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THE DESIGN OF

STEEL MILL BUILDINGS

AND

THE CALCULATION OF

STRESSES IN FRAMED STRUCTURES

BY

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PREFACE.

This book is intended to provide a short course in the calculation of stresses in framed structures and to give a brief discussion of mill building construction. The book is intended to supplement the elementary books on stresses on the one hand, and the more elaborate treatises on bridge design on the other. While the book is concerned chiefly with mill buildings it is nevertheless true that much of the matter will apply equally well to all classes of steel frame construction.

In the course in stresses an attempt has been made to give a concise, logical and systematic treatment. Both the algebraic and graphic methods of calculating stresses are fully described and illustrated. Each step in the solution is fully explained and analyzed so that the student will get a definite idea of the underlying principles.

Attention is called to the graphic solutions of the transverse bent, the portal and the two-hinged arch, which are believed to be new, and have proved their worth by actual test in the class room. The diagram for finding the stress in eye-bars due to their own weight is new, and its use will save considerable time in designing bridges.

In the discussion of mill building construction the aim has been to describe the methods of construction and the material used, together with a brief treatment of mill building design, and the making of estimates of weight and cost. The underlying idea has been to give methods, data and details not ordinarily available, and to discuss the matter presented in a way to assist the engineer in making his designs and the detailer in developing the designs in the drafting room. Every engineer should be familiar and be provided with one or more of the standard handbooks, and therefore only such tables as are not ordinarily available are given.

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The present book is a result of two years experience as designing engineer and contracting agent for the Gillette-Herzog Mfg. Co., Minneapolis, Minn., and four years experience in teaching the subject at the University of Illinois. This book represents the course given by the author in elementary stresses and in the design of metal structures, preliminary to a course in bridge design. While written primarily for the author's students it is hoped that the book will be of interest to others, especially to the younger engineers.

As far as practicable credit has been given in the body of the book for drawings and data. In addition the author wishes to acknowledge his indebtedness to various sources for drawings and information to which it has not been practicable to give proper individual credit. He wishes to thank Messrs. C. W. Malcolm, L. G. Parker and R. H. Gage, Instructors in Civil Engineering in the University of Illinois, for assistance in preparing the drawings, especially Mr. Malcolm who made a large part of the drawings.

The author will consider it a favor to have errors brought to his notice.

Champaign, Ill., M. S. K.
August 17, 1903.

PREFACE TO THE SECOND EDITION.

In this enlarged edition more than 100 pages of new material, including many cuts, have been added. The additions include a discussion of influence diagrams; the calculation of stresses in pins; a chapter on "Graphic Methods for Calculating the Deflection of Beams"; data on loads, foundations, and saw-tooth roofs; descriptions of various buildings; and an appendix giving the analyses of 22 Problems in Graphic Statics.

The author wishes to acknowledge the appreciation with which the first edition was received by engineers and instructors.

Boulder, Colo., M. S. K.
June 1, 1906.

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STEEL MILL BUILDINGS

INTRODUCTION.

Steel mill buildings may be divided into three classes as follows: (1) steel frame mill buildings; (2) steel mill buildings with masonry filled walls; and (3) mill buildings with masonry walls.

1. **Steel Frame Mill Buildings.**—A steel frame mill building is made by covering a self-supporting steel frame with a light covering, usually fireproof. The framework consists of transverse bents firmly braced by purlins, girts and diagonal braces. The usual methods of arranging the framework are as shown in Fig. 1.

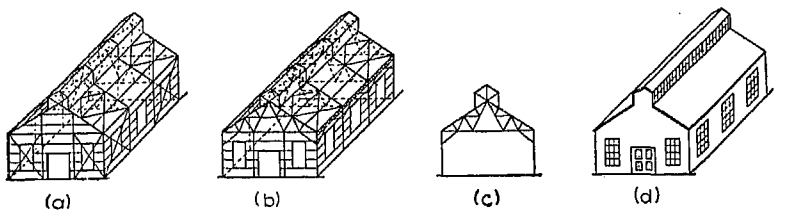


FIG. 1.

An intermediate transverse bent (c), Fig. 1, consists of a steel roof truss with its ends supported on steel posts, and is made rigid by knee braces. The posts are either supported on the foundations or are anchored by them. The end bents are made either by running the end posts up to the end rafters as in (a), or by means of an end trussed bent as in (b) Fig. 1. The end trussed bent (b) is usually preferred where extensions are contemplated, although the end post bent (a) is equally satisfactory and is usually somewhat cheaper.

The building is firmly braced transversely by means of bracing in the planes of the upper and lower chords and in the end bents, and longitudinally by means of bracing in the sides and in the planes of the upper and lower chords.

The roof covering is supported on steel purlins placed at right angles to the trusses and rafters. The side covering is fastened to horizontal girts which are fastened to the side and end posts. Where warmth is desired the roof and sides are lined.

Steel frame mill buildings are usually covered with corrugated iron or steel fastened to sheathing or directly to the purlins and girts. Expanded metal and plaster, or wire netting and plaster has been used to a limited extent for covering the sides and for sheathing the roof, and will certainly be much used in the future where permanent structures are required. In the latter case slate or tile roofing is commonly used.

The buildings are lighted by means of windows in the side walls and the clerestory of the monitor ventilator shown in Fig. 1, or by means of windows in the side walls and skylights in the roof. Ventilation is effected by means of the monitor ventilator shown in Fig. 1 or by means of circular ventilators. Where glass is used in the clerestory of monitor ventilators the sash are made movable. The glass in the clerestory of monitor ventilators is often replaced by louvres which allow a free circulation of air and keep out the storm. In foundries and smelters the clerestory is often left entirely open or is slightly protected by simple swinging shutters.

2. **Steel Mill Buildings with Masonry Filled Walls.**—In mill buildings of this type part of the bracing in the side walls is usually omitted and the space between the columns is filled with a light wall of brick, stone, concrete or hollow tile. The construction of the roof and other constructional details are essentially the same as for steel frame mill buildings. Buildings of this type are quite rigid and are usually somewhat cheaper than type (3).

3. **Mill Buildings with Masonry Walls.**—Buildings of this type are made by supporting the roof trusses directly on brick, stone or concrete walls. The construction of the roof is essentially the same as for types (1) and (2), except that the trusses are somewhat lighter on account of the smaller wind stresses.

The discussion of the simple steel frame mill building shown in Fig. 1 includes practically all the problems and details which are encountered in the design of steel mill buildings of all types.

The problems involved in the design of mill buildings will be divided into Part I, Loads; Part II, Stresses; Part III, Design of Mill Buildings; and Part IV, Miscellaneous Structures. In general the discussion will relate to the design of mill buildings but in a few cases, particularly in stresses, quite a number of problems will be discussed that are only indirectly related to the subject.

PART 1.

LOADS.

The loads to be provided for in designing a mill building will depend to a large degree upon the use to which the finished structure is to be put. The loads may be classed under (1) dead loads; (2) snow loads; (3) wind loads; and (4) miscellaneous loads. Concentrated floor and roof loads, girder and jib crane, and miscellaneous loads should receive special attention, and proper provision should be made in each case. No general solution can be given for providing for miscellaneous loads, but each problem must be worked out to suit local conditions.

CHAPTER I.

Dead Loads.

Dead loads may be divided into (a) weight of structure; (b) concentrated loads.

The weight of the structure may be divided into (1) the weight of the roof trusses; (2) the weight of the roof covering; (3) the weight of the purlins and bracing; (4) the weight of the side and end walls. The first three items, together with the concentrated roof loads, constitute the dead loads used in designing the trusses.

The weights of mill buildings vary so much that it is not possible to give anything more than approximate values for the different items which go to make up the dead load. The following data will, however, materially assist the designer in arriving at approximately the proper

dead load to assume for computing stresses, and the approximate weight of metal to use as a basis for preliminary estimates.

Weight of Roof Trusses.—The weight of roof trusses varies with the span, the distance between trusses, the load carried or capacity of the truss, and the pitch.

The empirical formula

$$W = \frac{P}{45} A L \left(1 + \frac{L}{5\sqrt{A}}\right) \qquad (1)$$

where

W = weight of steel roof truss in pounds;

P = capacity of truss in pounds per square foot of horizontal projection of roof (30 to 80 lbs.);

A = distance center to center of trusses in feet (8 to 30 feet);

L = span of truss in feet;

was deduced by the author from the computed and shipping weights of mill building trusses. The trusses were riveted Fink trusses with purlins placed at panel points, and were made up of angles with connecting plates; minimum size of angles 2" × 2" × ¼", minimum thickness of plates ¼".

The trusses whose weights were used in deducing this formula had a pitch of ¼ (6" in 12"), but the formula gives quite accurate results for trusses having a pitch of ⅛ to ½. The trusses were designed for a tensile stress of 15000 lbs. per square inch and a compressive stress of $15000 - 55\frac{l}{r}$ lbs. per square inch, where l = length and r the radius of gyration of the member, both in inches.

The weight of steel roof trusses for a capacity, P, of 40 lbs. per square foot for different spacings is given in Fig. 2. The weights of trusses for other capacities can be obtained by multiplying the tabular values by the ratio of the capacities.

Dividing (1) by $A L$ we have the weight of roof truss, W_s, per square foot of horizontal projection of the roof

$$W_s = \frac{P}{45}\left(1 + \frac{L}{5\sqrt{A}}\right) \qquad (2)$$

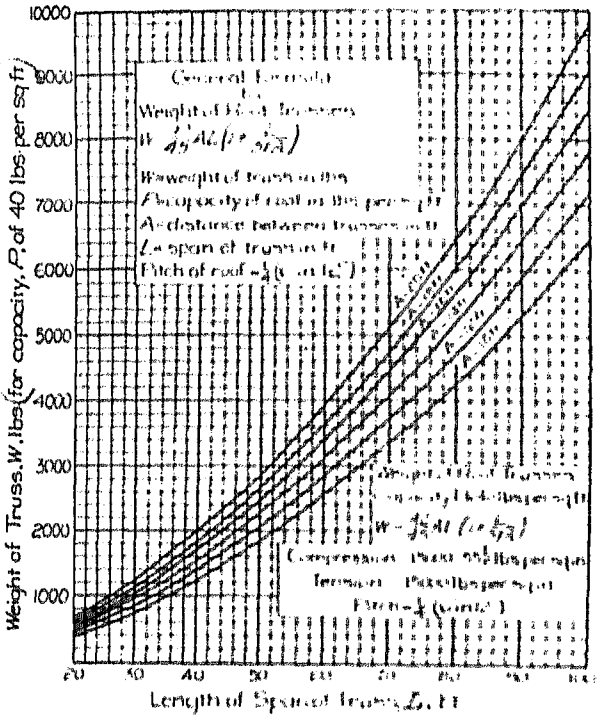
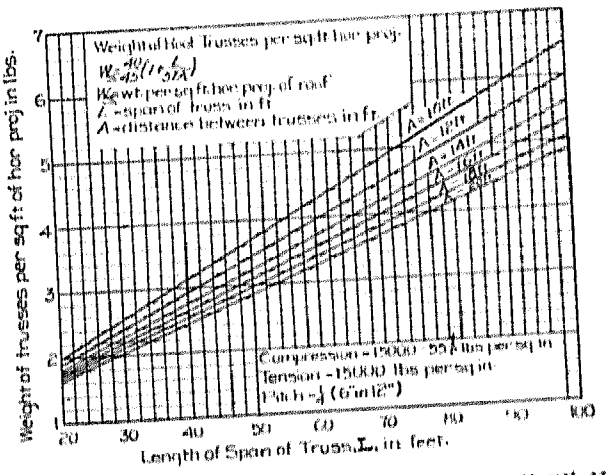


FIG. 2. WEIGHT OF ROOF TRUSSES FOR A CAPACITY OF 40 LBS. PER SQUARE FOOT.

The weight of steel roof trusses per square foot of horizontal projection of roof for a capacity, P, of 40 lbs. per square foot is given in Fig 3.

It should be noted that W_n is the dead load per square foot carried by an interior truss. The actual weight of trusses per square foot of horizontal projection for a building with n panels will be $W'_n \frac{(n-1)}{n}$ where end post bent (a), Fig. 1, is used, and $W'_n \frac{(n+1)}{n}$ where end truss bent (b), Fig. 1, is used, assuming that all trusses are made alike.

Weight of Light Trusses. - Formula (1) gives the weight of mill building trusses and will usually cover the weight of knee braces and ventilator framing. By reducing the minimum thickness of metal and



3. WEIGHT OF ROOF TRUSSES PER SQUARE FOOT OF HORIZONTAL PROJECTION, FOR A CAPACITY OF 40 LBS. PER SQUARE FOOT.

kinning the sections it is possible to materially reduce the weights.

Weight of Simple Roof Trusses.—Simple roof trusses supported

TABLE I.

WEIGHT OF FINK TRUSSES, SUPPORTED ON MASONRY WALLS, DESIGNED FOR A VERTICAL LOAD OF 40 POUNDS PER SQUARE FOOT OF HORIZONTAL PROJECTION OF ROOF.

pan, L, Feet	Distance between Trusses, A, in Feet	Weight of Truss, W, in Pounds	Span, L, in Feet	Distance between Trusses, A, in Feet	Weight of Truss, W, in Pounds
30	16	741	65	20	3226
30	14	621	70	20	3951
35	16	910	75	20	4564
40	16	1211	75	14	3200
40	14	976	80	20	5160
45	16	1423	85	25	6730
50	16	1865	85	14	4080
50	14	1550	90	25	8010
55	16	2103	95	25	8600
60	20	2870	100	25	9312
60	14	2120			

on walls will usually weigh somewhat less than the value given by Formula (1). The computed weights of Fink roof trusses without ventilators and with purlins spaced from 4 to 8 feet are given in Table I.

These trusses were designed by two different bridge companies to serve as standards and represent minimum weights. The trusses with a spacing of 14 feet were designed with minimum thickness of metal 3-16" and minimum size of angles 2" × 1⅜" × 3-16". In the remainder of the trusses the minimum thickness of plates was ¼" and minimum size of angles 2" × 2" × ¼". The trusses are all too light to give good service although their use in temporary structures may sometimes be allowable.

Weight of Purlins, Girts, Bracing, and Columns.—Steel purlins will weigh from 1½ to 4 pounds per square foot of area covered, depending upon the spacing and the capacity of the trusses and the snow load. If possible the actual weight of the purlins should be calculated. Girts and window framing will weigh from 1¼ to 3 pounds per square foot of net surface. Bracing is quite a variable quantity. The bracing in the planes of the upper and lower chords will vary from ½ to 1 pound per square foot of area. The side and end bracing, eave struts and columns will weigh about the same per square foot of surface as the trusses.

Weight of Covering.—The weight of corrugated iron or steel covering varies from 1½ to 3 pounds per square foot of area.

WEIGHT OF FLAT AND CORRUGATED STEEL SHEETS WITH 2½ INCH CORRUGATIONS.

Gage No.	Thickness in inches	Weight per Square (100 sq. ft.)			
		Flat Sheets		Corrugated Sheets	
		Black	Galvanized	Black Painted	Galvanized
16	.0625	250	266	275	291
18	.0500	200	216	220	236
20	.0375	150	166	165	182
22	.0313	125	141	138	154
24	.0250	100	116	111	127
26	.0188	75	91	84	99
28	.0156	63	79	69	86

In estimating the weight of corrugated steel allow about 25 per cent for laps where two corrugations side lap and 6 inches end lap are required, and about 15 per cent for laps where one corrugation side lap and 4 inches end lap are required. Nos. 20 and 22 corrugated steel are commonly used on the roof and Nos. 22 and 24 on the sides.

Weight of Roof Covering.—The approximate weight per square foot of various roof coverings is given in the following table:

Corrugated iron, without sheathing	1 to	3	lbs.
Felt and asphalt, without sheathing		2	"
Felt and gravel, without sheathing	8 to	10	"
Slate, 3-16" to ¼", without sheathing	7 to	9	"
Tin, without sheathing	1 to	1½	"
Skylight glass, 3-16" to ½", including frames	4 to	10	"
White pine sheathing 1" thick		3	"
Yellow pine sheathing 1" thick		4	"
Tiles, flat	15 to	20	"
Tiles, corrugated	8 to	10	"
Tiles, on concrete slabs	30 to	35	"
Plastered ceiling		10	"

For additional data on weight of roof coverings, see Chapter XIX. The actual weight of roof coverings should be calculated if possible.

Weight of the Structure.—The weight of the roof can now be found. The weight of the steel in the sides and ends is approximately the same per square foot as the steel work in the roof.

A very close approximation to the weight of the steel in the entire structure where no sheathing is used and the same weight of corrugated iron is used on sides as on roof, may be found as follows: Take the sum of the horizontal projection of the roof and the net surface of the sides and ends, after subtracting one-half of the area of the windows, wooden doors and clear openings; multiply the sum of these areas by the weight per square foot of the horizontal projection of the roof, and the product will be the approximate weight of the steel in the structure.

CHAPTER II.

Snow Loads.

The annual snowfall in different localities is a function of the humidity and the latitude and is quite a variable quantity. The amount of snow on the ground at one time is still more variable. In the Lake Superior region very little of the snow melts as it falls, and almost the entire annual snowfall is frequently on the ground at one time; while on the other hand in the same latitude in the Rocky Mountains the dry winds evaporate the snow in even the coldest weather and a less proportion accumulates. In latitudes of 35 to 45 degrees the heavy snowfalls are often followed by a sleeting rain, and the snow and ice load on roofs sometimes nearly equals the weight of the annual snowfall.

From the records of the snowfall for the past ten years as given in the reports of the U. S. Weather Bureau and data obtained by personal experience, in British Columbia, Montana, the Lake Superior region and central Illinois the author presents the values given in Fig. 4 for snow loads for roofs of different inclinations in different latitudes. For the Pacific coast and localities with low humidity, take one-half of the values given. The weight of newly fallen snow was taken at 5 lbs. and packed snow at 12 lbs. per cubic foot.

A high wind may follow a heavy sleet and in designing the trusses the author would recommend the use of a minimum snow and ice load as given in Fig. 4 for all slopes of roofs. The maximum stresses due to the sum of this snow load, the dead and wind loads; the dead and the wind loads; or of the maximum snow load and the dead load being used in designing the members.

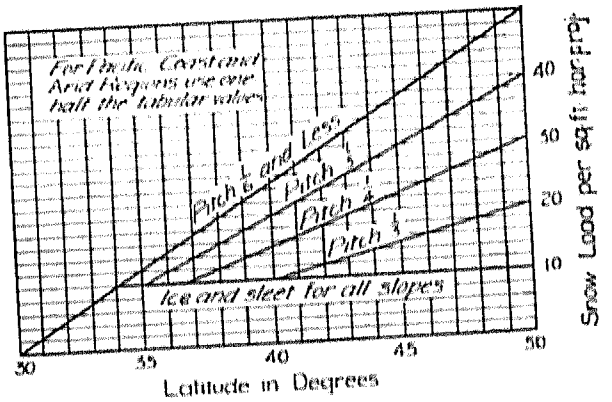


FIG. 4. SNOW LOAD ON ROOFS FOR DIFFERENT LATITUDES, IN LBS. PER SQUARE FOOT.

Snow loads per square foot of horizontal projection of roof are specified in various localities as follows: Chicago and New York, 20 lbs.; Cincinnati and St. Louis, 10 lbs.; New England, 30 lbs. The Baltimore and Ohio Railroad specifies 20 lbs per square foot of horizontal projection of roof.

CHAPTER III.

Wind Loads.

Wind Pressure.—The wind pressure (P) in pounds per square foot on a flat surface normal to the direction of the wind for any given velocity (V) in miles per hour is given quite accurately by the formula

$$P = 0.004\, V^2 \qquad (3)$$

The following table gives the pressure per square foot on a flat surface normal to the direction of the wind for different velocities as calculated by formula (3).

Vel. in miles per hour.	Pressure, lbs. per square foot.	
10	0.4	Fresh breeze.
20	1.6	
30	3.6	Strong wind.
40	6.4	High wind.
50	10.0	Storm.
60	14.4	Violent storm.
80	25.6	Hurricane.
100	40.0	Violent hurricane.

The pressure on other than flat surfaces may be taken in per cents of that given by formula (3) as follows: 80 per cent on a rectangular building; 60 per cent on the convex side of cylinders; 115 to 130 per

cent on the concave side of cylinders, channels and flat cups; and 130 to 170 per cent on the concave sides of spheres and deep cups.

The pressure on the vertical sides of buildings is usually taken at 30 pounds per square foot, equivalent to P equals 37½ pounds in formula (3). This would give a velocity of 96 miles per hour, which would seem to be sufficient for all except the most exposed positions. The velocity of the wind in the St. Louis tornado was about 120 miles per hour. The records of the U. S. Weather Bureau for the last ten years show only one instance where the velocity of the wind as recorded by the anemometer was more than 90 miles per hour. The actual pressure of wind gusts has been found to be about 60 per cent and the actual steady wind pressure only about 36 per cent of that registered by ordinary small anemometers, which further reduces the intensity of the observed pressures. The wind pressure has been found to increase as the distance above the ground increases.

Recent German specifications for design of tall chimneys specify wind loads per square foot as follows: 26 pounds on rectangular chimneys; 67 per cent of 26 pounds on circular chimneys; and 71 per cent of 26 pounds on octagonal chimneys.

The building laws of New York, Boston and Chicago require that steel buildings be designed for a horizontal wind pressure of 30 pounds per square foot. The Baltimore and Ohio Railroad specifies a horizontal wind pressure of 30 pounds per square foot.

From the above discussion it would seem that 30 pounds per square foot on the sides and the normal component of a horizontal pressure of 30 pounds on the roof would be sufficient for all except exposed locations. If the building is somewhat protected a horizontal pressure of 20 pounds per square foot on the sides is certainly ample for heights less than, say, 30 feet.

Wind Pressure on Inclined Surfaces.—The wind is usually taken as acting horizontally and the normal component on inclined surfaces is calculated.

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The normal component of the wind pressure on inclined surfaces has usually been computed by Hutton's empirical formula

$$P_n = P(\sin A)\ 1.842 \cos A - 1 \tag{4}$$

where P_n equals the normal component of the wind pressure, P equals the pressure per square foot on a vertical surface, and A equals the angle of inclination of the surface with the horizontal, Fig. (5).

The formula due to Duchemin

$$P_n = P \frac{2 \sin A}{1 + \sin^2 A} \tag{5}$$

where P_n, P and A are the same as in (4), gives results considerably larger for ordinary roofs than Hutton's formula, and is coming into quite general use.

The formula

$$P_n = \frac{P}{45} A \tag{6}$$

where P_n and P are the same as in (4) and (5), and A is the angle of inclination of the surface in degrees (A being equal to or less than 45°), gives results which agree very closely with Hutton's formula, and is much more simple.

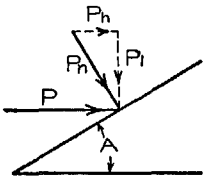


FIG. 5.

Hutton's formula (4) is based on experiments which were very crude and probably erroneous. Duchemin's formula (5) is based on very careful experiments and is now considered the most reliable formula in use. The Straight Line formula (6) agrees with experiments quite closely and is preferred by many engineers on account of its simplicity.

The values of P_n as determined by Hutton's, Duchemin's and the

Straight Line formulas are given in Fig. 6, for P equals 20, 30 and 40 pounds.

It is interesting to note that Duchemin's formula with P equals 30 pounds gives practically the same values for roofs of ordinary inclination as is given by Hutton's and the Straight Line formulas with P equals 40 pounds.

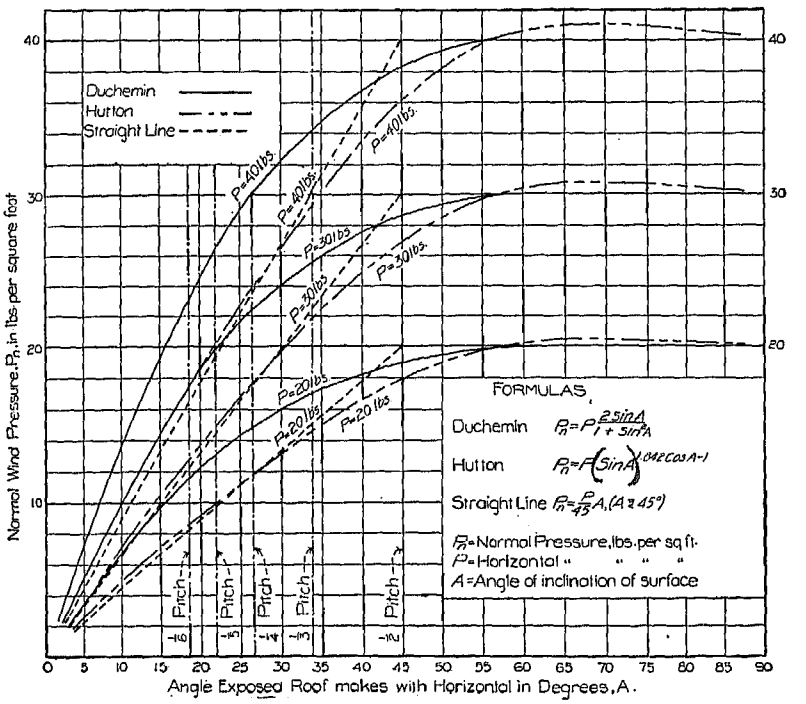


FIG. 6. NORMAL WIND LOAD ON ROOF ACCORDING TO DIFFERENT FORMULAS.

Duchemin has also deduced the formula

$$P_h = P\,\frac{2\sin^2 A}{1+\sin^2 A} \tag{7}$$

where P_h in (7) equals the pressure parallel to the direction of the wind, Fig. 5; and

$$P_l = P\,\frac{2\sin A\,\cos A}{1+\sin^2 A} \tag{8}$$

where P_h in (8) equals the pressure at right angles to the direction of the wind, Fig. 5. P_l may be an uplifting, a depressing or a side pressure. With an open shed in exposed positions the uplifting effect of the wind often requires attention. In that case the wind should be taken normal to the inner surface of the building on the leeward side, and the uplifting force determined by using formula (8). If the gables are closed a deep cup is formed, and the normal pressure should be increased 30 to 70 per cent.

That the uplifting force of the wind is often considerable in exposed localities is made evident by the fact that highway bridges are occasionally wrecked by the wind. The most interesting example known to the author is that of a 100-foot span combination bridge in Northwestern Montana which was picked up bodily by the wind, turned about 90 degrees in azimuth and dropped into the middle of the river. The end bolsters were torn loose although drift-bolted to the abutments.*

The wind pressure is not a steady pressure, but varies in intensity, thus producing excessive vibrations which cause the structure to rock if the bracing is not rigid. The bracing in mill buildings should be designed for initial tension, so that the building will be rigid. Rigidity is of more importance than strength in mill buildings.

For further information on this subject see a very elaborate and valuable monograph on "Wind Pressures in Engineering Construction," by Capt. W. H. Bixby, M. Am. Soc. C. E., published in Engineering News, Vol. XXXIII., pp. 175–184, March, 1895.

Wind Pressure on Office Buildings.—The following specification for wind pressure on office buildings has been proposed by Mr. C. C. Schneider.†

Wind Pressure.—The wind pressure shall be assumed at 30 lb. per sq. ft. acting in either direction horizontally:

1. On the sides and ends of buildings and on the actually exposed surface, or the vertical projection of roofs;
2. On the total exposed surfaces of all parts composing the metal framework. The framework shall be considered an independent structure, without walls, partitions or floors.

* For a description of the wreck by the wind of the High Bridge over the Mississippi River at St. Paul, Minn., see article by C. A. P. Turner in Trans. Am. Soc. C. E., Vol. 54, p. 31.

† Trans. Am. Soc. C. E., Vol. 54, 1905.

CHAPTER IV.

Miscellaneous Loads.

LIVE LOADS ON FLOORS.—Live loads on floors for mill buildings are very hard to classify and should be calculated for each case.

Floor loads as specified in the building laws of various cities are given in Table II, and the engineer should govern himself accordingly.

TABLE II.

Floor loads in pounds per square foot as specified in various cities.

	New York	Chicago	Philadelphia	Boston
Dwellings	Upper floors 75 1st floor .50	100	100	100
Public Buildings..	90	100	120	150
Light Manufacturing	120	100	100	.....
Warehouses and Factories.....	150 and up	.....	150 and up	250 and up
Sidewalks	300	.....	.....	.....

Without reference to building laws the live loads per square foot, exclusive of weight of floor materials, given below are about standard practice.

Dwellings ..... 70 lbs. per sq. ft.
Offices ..... 70 to 100 lbs. per sq. ft.
Assembly halls .....120 to 150 lbs. per sq. ft.
Warehouses .....250 and up, lbs. per sq. ft.
Factories .....200 to 450 lbs. per sq. ft.

The weight of floors above ground in mill buildings varies so much that it is useless to give weights. For a few data on weights of floors see Chapter on Floors.

WEIGHT OF HAND CRANES.—The approximate weight of a few of the common sizes of hand cranes made by Pawling and Harnischfeger, Milwaukee, Wis., and the maximum load on each wheel when the loaded trolley is at one end is given in Table III.

TABLE III.

WEIGHT OF TRAVELING HAND CRANES.

Capacity of Crane in Tons	Distance c to c of end wheels	20-FOOT SPAN		30-FOOT SPAN	
		Weight of Crane lbs.	Maximum Load on each Wheel lbs.	Weight of Crane lbs.	Maximum Load on each Wheel lbs.
3	3′ — 0″	4500	4500	5000	5000
5	3′ — 0″	5500	7000	6000	7500
7½	3′ — 8″	8000	10500	9000	11500
10	8′ — 0″	15000	18000	17000	20000
15	8′ — 0″	16000	20000	18000	21000
20	8′ — 6″	20000	26000	22000	27000

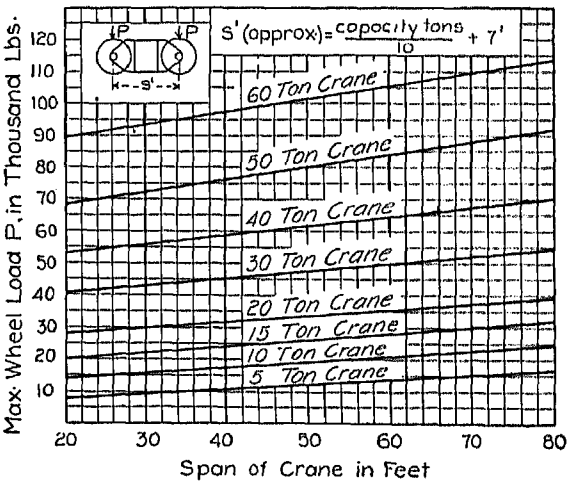


FIG. 7.

WEIGHT OF ELECTRIC CRANES.—The maximum load on each of the end wheels for common sizes of electric cranes made by

Pawling and Harnischfeger is given in Fig. 7. Cranes made by different manufacturers differ considerably in weight.

The weights and dimensions of typical traveling cranes as given in Table IIIa have been proposed for adoption as a standard.

TABLE IIIa.

TYPICAL ELECTRIC TRAVELING CRANES.*

Capacity in Tons.	Span.	Wheel Base.	Maximum Wheel Load, in Pounds.	s.	v.	Weight of Rail for: Plate Girders.	Beams.
5	40	8 ft. 6 in.	12,000	10 in.	7 ft.	40 lb. per yd.	40
	60	9 " 0 "	13,000	"	"	40 "	40
10	40	9 " 0 "	19,000	"	"	45 "	40
	60	9 " 6 "	21,000	"	"	45 "	40
15	40	9 " 6 "	26,000	"	"	50 "	50
	60	10 " 0 "	29,000	"	"	50 "	50
20	40	10 " 0 "	33,000	12 in.	8 ft.	55 "	50
	60	10 " 6 "	36,000	"	"	55 "	50
25	40	10 " 0 "	40,000	"	"	60 "	50
	60	10 " 6 "	44,000	"	"	60 "	50
30	40	10 " 6 "	48,000	"	"	70 "	60
	60	11 " 0 "	52,000	"	"	70 "	60
40	40	11 " 0 "	64,000	14 in.	9 ft.	80 "	60
	60	12 " 0 "	70,000	"	"	80 "	60
50	40	11 " 0 "	72,000	"	"	100 "	60
	60	12 " 0 "	80,000	"	"	100 "	60

1. Wheel-load can be assumed as distributed in top flange, over a distance equal to depth of girder, with a maximum limit of 30 in.

2. In addition to the vertical load, the top flanges of the girder shall withstand a lateral loading of two-tenths of the lifting capacity of the crane, equally divided between the four wheels of the crane.

s = Side clearance from center of rail.

v = Vertical clearance from top of rail.

3. The top flanges of the crane girders shall not be of smaller width than one-twentieth of their unsupported length.

WEIGHTS OF MISCELLANEOUS MATERIAL.—The weights of various kinds of merchandise are given in Table IV. For weights of other materials consult steel makers hand books.

* Mr. C. C. Schneider in Trans. Am. Soc. C. E., Vol. 54, 1905.

TABLE IV.

WEIGHTS OF MERCHANDISE.*

Commodity	Weight in lbs. per cubic foot	Commodity	Weight in lbs. per cubic foot
Wool in Bales	5 to 28	Caustic Soda	88
Woolen Goods	13 to 22	Barrel Starch	23
Baled Cotton	12 to 43	Barrel Lime	50
Cotton Goods	11 to 37	" Cement	73
Rags in Bales	7 to 36	" Plaster	53
Paper	10 to 69	" Lard Oil	34
Wheat	39 to 44	Rope	42
Corn	31	Box Tin	278
Oats	27	Box Glass	60
Baled Hay and Straw	14 to 19	Crate Crockery	40
Bleaching Powder	31	Bale Leather	16 to 23
Soda Ash	62	Sugar	45
Box Indigo	43	Cheese	30

The weights of miscellaneous building materials are given in Table IVa.

TABLE IVa.

WEIGHTS OF BUILDING MATERIALS.

Material.	Weight lbs. per cu. ft.	Material.	Weight lbs. per cu. ft.
Paving brick	150	Glass	160
Common building brick	120	Snow, freshly fallen	10
Soft building brick	100	Snow, wet	50
Granite	170	Spruce	25
Marble	170	Hemlock	25
Limestone	160	White pine	25
Sandstone	145	Douglas fir	30
Slag	40	Yellow pine	40
Gravel	120	White oak	50
Slate	175	Common brickwork	100–120
Sand, clay and earth (dry)	100	Rubble masonry	130–150
Sand, clay and earth (wet)	120	Ashlar masonry	140–160
Mortar	100	Cast iron	450
Stone concrete	130–150	Wrought iron	480
Cinder concrete	70	Steel	490
Paving asphaltum	100	Plaster, ceiling	10 to 15 lb. per sq. ft.
Plaster of paris	140		

*From Report V. Insurance Engineering Experiment Station. Edward Atkinson, Director, Boston, Mass.

CONCENTRATED LIVE LOADS.—The loads given in Table IIa have been proposed for different classes of buildings.* These loads consist of:

(*a*) A uniform load per square foot of floor area;

(*b*) A concentrated load which shall be applied to all points of the floor;

(*c*) A uniform load per linear foot for girders.

The maximum result is to be used in calculations.

The specified concentrated loads shall also apply to the floor construction between the beams for a length of five feet.

TABLE IIa.

LIVE LOADS FOR FLOORS.

Classes of Buildings.	Live Loads, in Pounds.		
	Distributed Load.	Concentrated Load.	Load per Linear Foot of Girder.
Dwellings, hotels, apartment-houses, dormitories, hospitals	40	2,000	500
Office buildings, upper stories	50	5,000	1,000
Schoolrooms, theater galleries, churches	60	5,000	1,000
Ground floors of office buildings, corridors and stairs in public buildings	80	5,000	1,000
Assembly rooms, main floors of theaters, ballrooms, gymnasia, or any room likely to be used for drilling or dancing	floor 100 columns 50	5,000	1,000
Ordinary stores and light manufacturing, stables and carriage-houses	80	8,000	1,000
Sidewalks in front of buildings	300	10,000	1,000
Warehouses and factories	from 120 up	Special.	Special.
Charging floors for foundries	" 300 "	"	"
Power-houses, for uncovered floors	" 200 "	The actual weights of engines, boilers, stacks, etc., shall be used, but in no case less than 200 lb. per sq. ft.	

If heavy concentrations, like safes, armatures, or special machinery, are likely to occur on floors, provision should be made for them.

* Mr. C. C. Schneider in Trans. Am. Soc. C. E., Vol. 54, 1905.

PART II.

STRESSES.

CHAPTER V.

Graphic Statics.

Equilibrium.—Statics considers forces as at rest and therefore in equilibrium. To have static equilibrium in any system of forces there must be neither translation nor rotation and the following conditions must be fulfilled for coplanar forces (forces in one plane).

$$\Sigma \text{ horizontal components of forces} = 0 \qquad \text{(a)}$$

$$\Sigma \text{ vertical components of forces} = 0 \qquad \text{(b)}$$

$$\Sigma \text{ moments of forces about any point} = 0 \qquad \text{(c)}$$

Representation of Forces.—A force is determined when its magnitude, line of action, and direction are known, and it may be represented graphically in magnitude by the length of a line, in line of action by the position of the line, and in direction by an arrow placed on the line, pointing in the direction in which the force acts. A force may be considered as applied at any point in its line of action.

Force Triangle.—The resultant, R, of the two forces P_1 and P_2 meeting at the point a in Fig. 8 is represented in magnitude and direction by the diagonal, R, of the parallelogram $a\ b\ c\ d$. The combining of the two forces P_1 and P_2 into the force R is termed composition of forces. The reverse process is called resolution of forces.

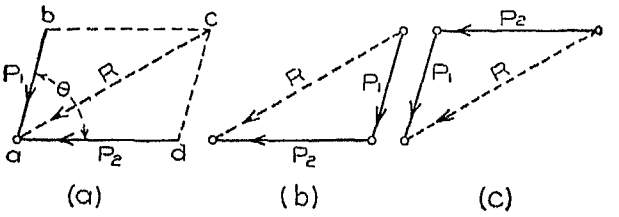


FIG. 8.

The value of R may also be found from the equation

$$R^2 = P_1^2 + P_2^2 + 2\,P_1\,P_2 \cos\theta$$

It is not necessary to construct the entire force parallelogram as in (a) Fig. 8, the force triangle (b) below or (c) above the resultant R being sufficient.

If only one force together with the line of action of the two others be given in a system containing three forces in equilibrium, the magnitude and direction of the two forces may be found by means of the force triangle.

If the resultant R in Fig. 8 is replaced by a force E equal in amount but opposite in direction, the system of forces will be in equilibrium, (a) or (b) Fig. 9. The force E is the equilibrant of the system of forces P_1 and P_2.

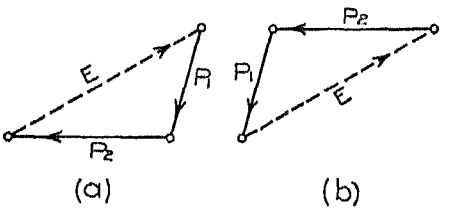
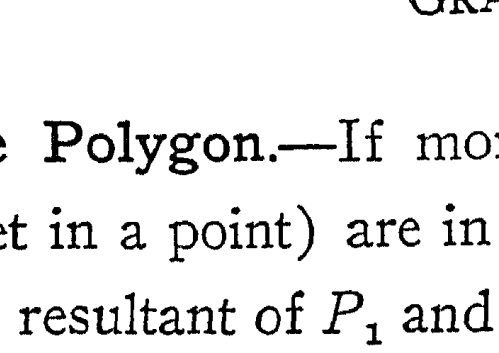


FIG. 9.

It is immaterial in what order the forces are taken in constructing the force triangle, as in Fig. 9, as long as the forces all act in the same direction around the triangle. The force triangle is the foundation of the science of graphic statics.

Force Polygon.—If mo
which meet in a point) are in
will be the resultant of P_1 and



(a)

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Equilibrium of Non-concurren

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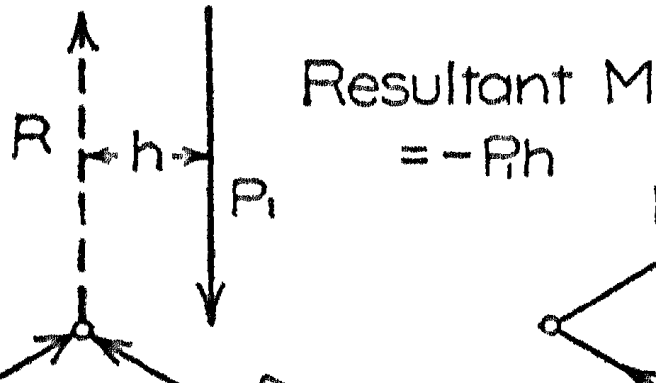

graphically by twice the area
as a base and the arm of the
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area of a triangle as shown in

It will be seen from the p
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resultant of all the forces save
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Equilibrium Polygon.—
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a force polygon and (b) is called a f
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system of forces is given by the clo
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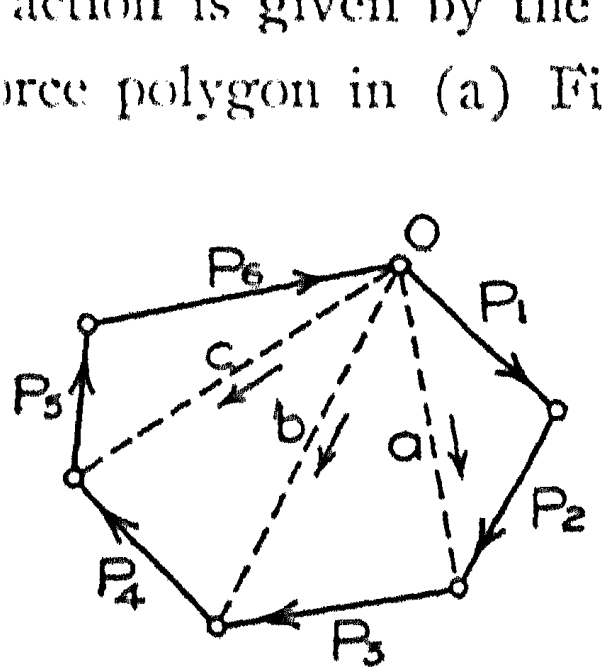
The force polygon in (a) Fig.



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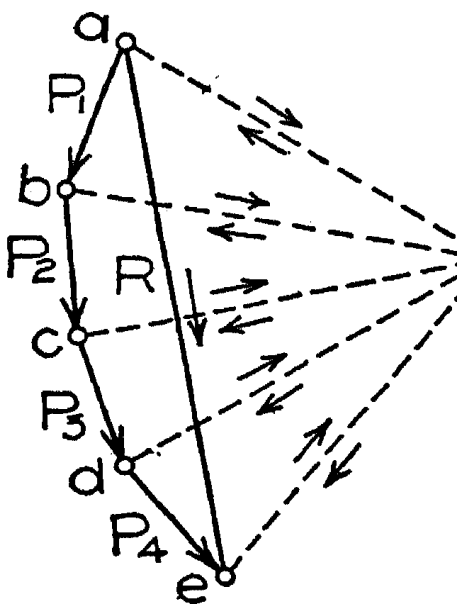

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It is immaterial in what orde
the force polygon, as long as the
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of R_2 must coincide and be e
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VIII).

Reactions of a Cantilev
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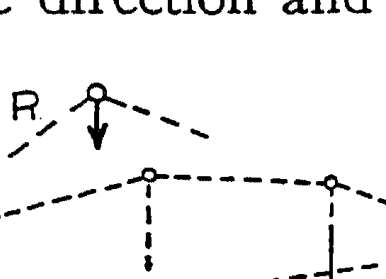

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The reactions R_1 and R_2 are therefor
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Equilibrium Polygon as a Fr

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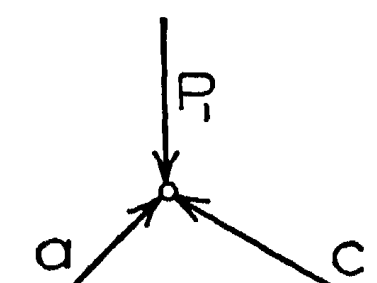

Graphic Moments.—In Fig
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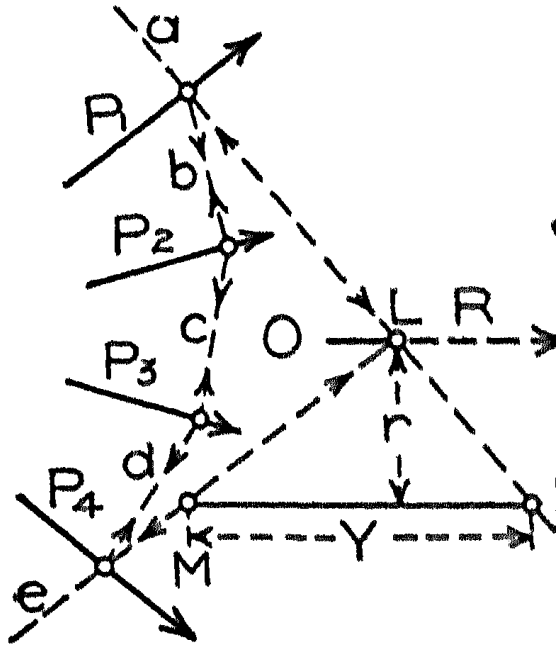

(a)

the line $M\,N = Y$ parallel to the

Bending Moments in a Bea

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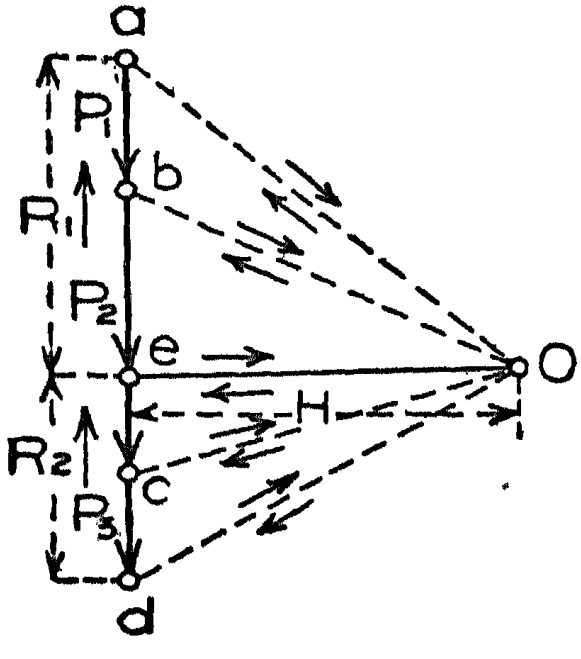

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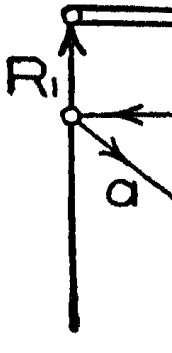

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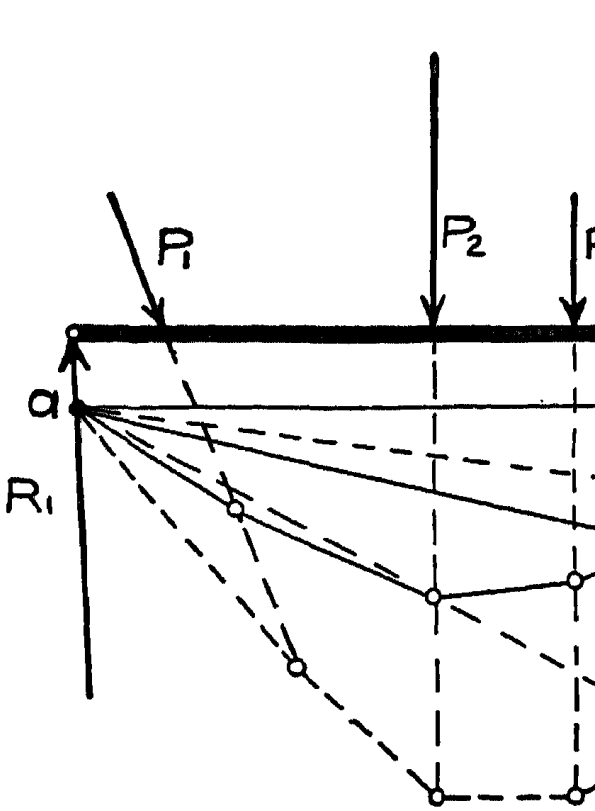


(a)

left of b, acting throug
a b'' parallel to closing

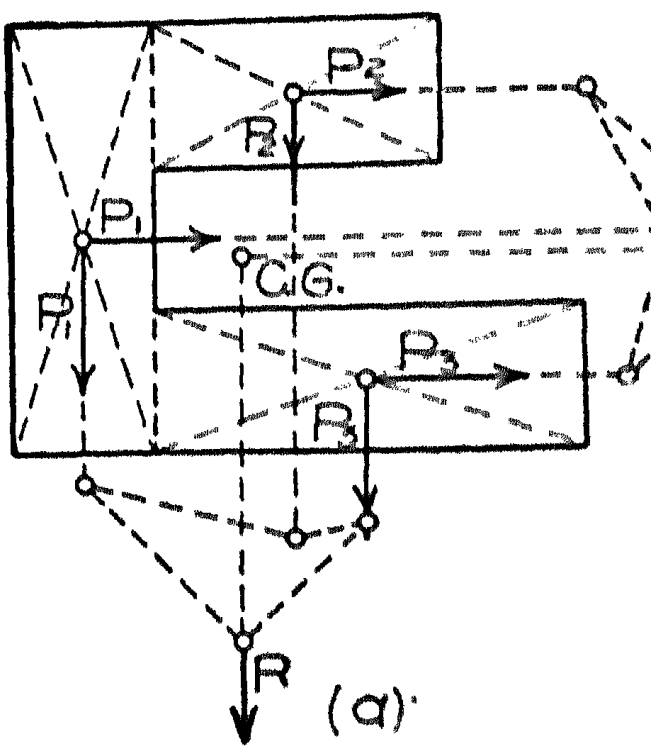
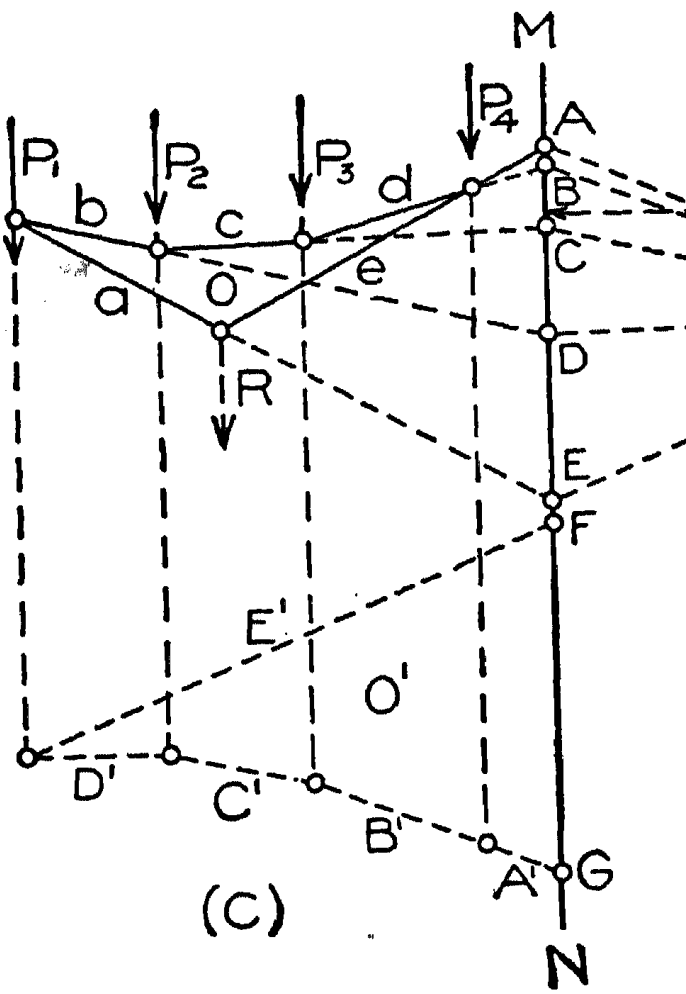


FIG.

Moment of Inertia of Forces.



(C)

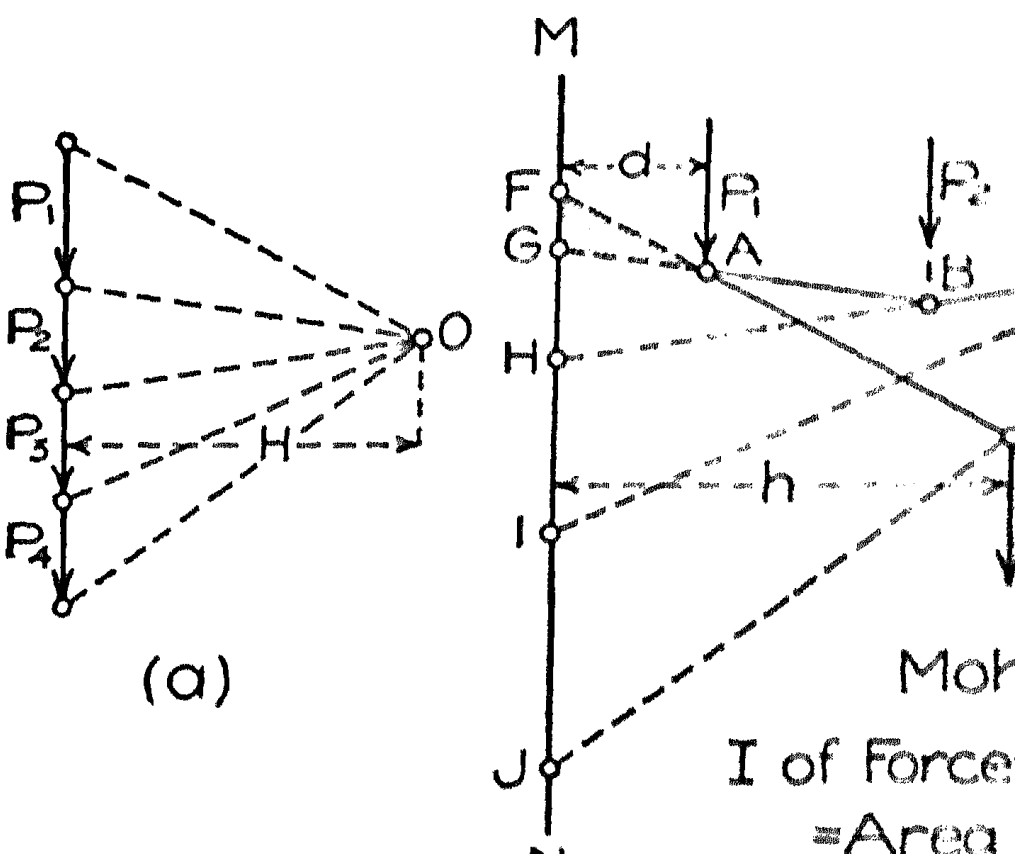


FIG. 23.

tem of forces equal to the area of the equil

Moment of Inertia of A

area about an axis in the sam
products of the differential
squares of the distances of th

The moment of inertia
through center of gravity of th
axis, and is the moment of ine
flexure in beams

where

M = bending moment at

S = extreme fibre stress

I = moment of inertia o

CHAPTER VI.

Stresses in Framed St

Methods of Calculation.—The determ
simple framed structures usually requires t
mental equations of equilibrium

Σ horizontal components of

Σ vertical components of for

Σ moments of forces about an

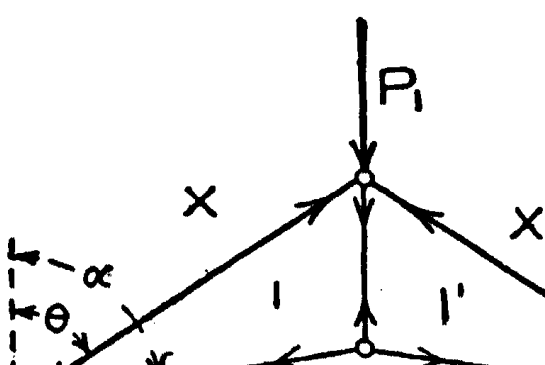
Algebraic Resolution.—I
algebraic resolution, the fund
translation

Σ horizontal co

Σ vertical comp

are applied (a) to each joint, o
side of a section cut through

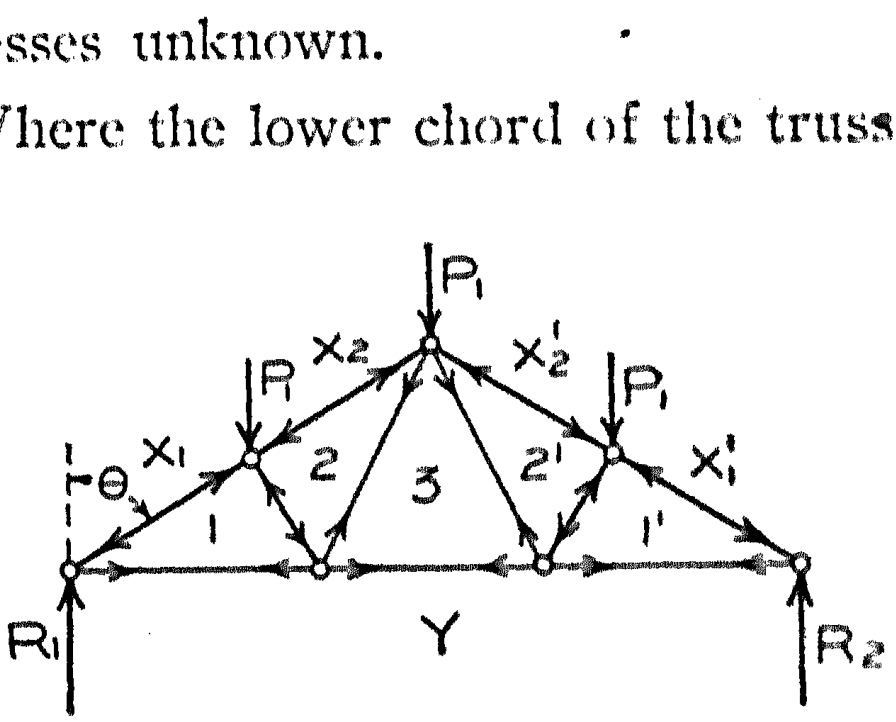
(a) *Forces at a Joint.*—
stresses in the members of the



It will be seen from the foregoing that
resolution consists in applying the principle o
external forces and internal stresses at each j

Since we have only two fundamental
(resolution) we can not solve a joint if there
or stresses unknown.

Where the lower chord of the truss is h

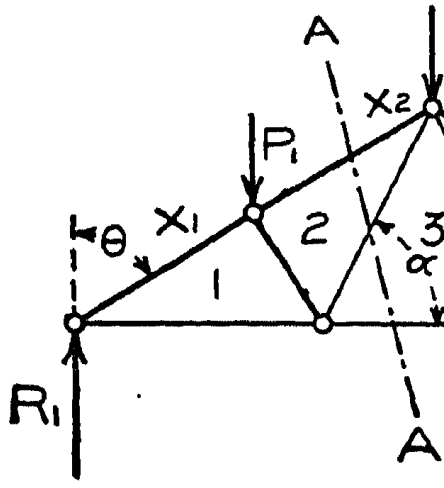


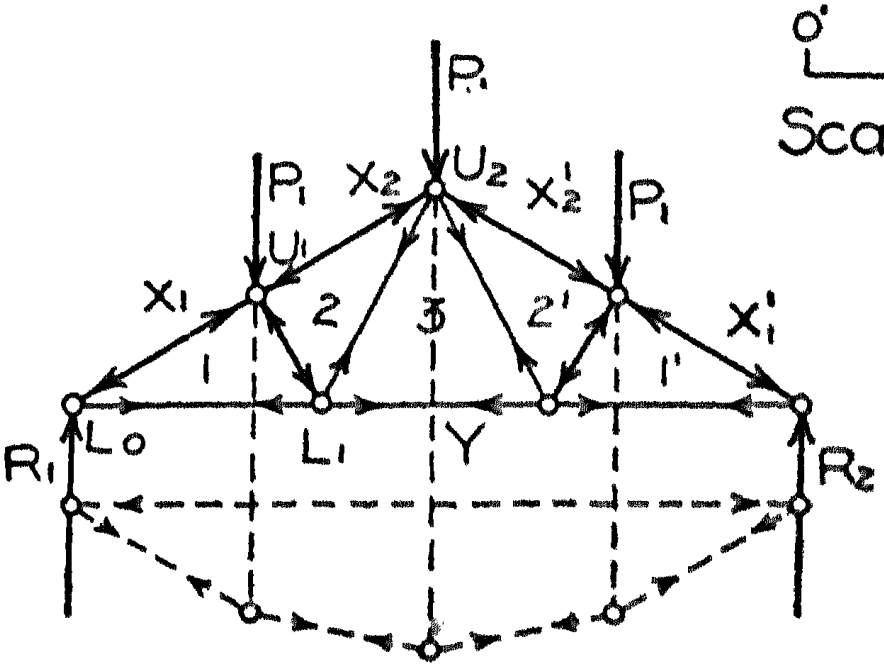
Applying equations (

$$3\text{-}y + 2\text{-}3$$

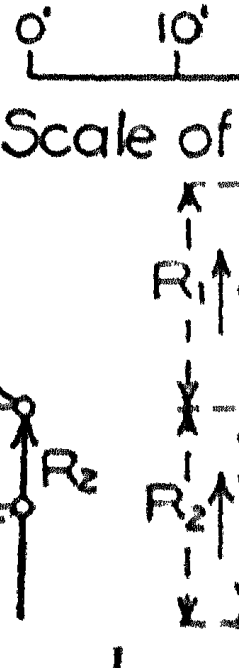
$$2\text{-}3 \sin \alpha$$

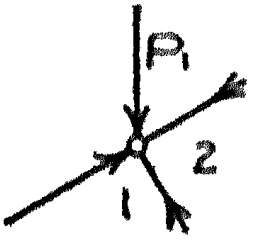
Now, if all but two
knowns may be found by



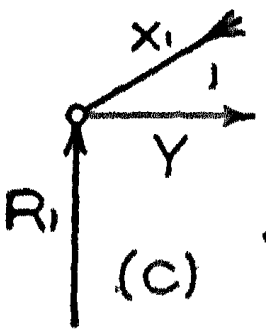

P1
P1
X2
U2
X'2
P1
U1
X1
2
3
2'
X'1
1
1'
R1
L0
L1
Y
R2

(a)

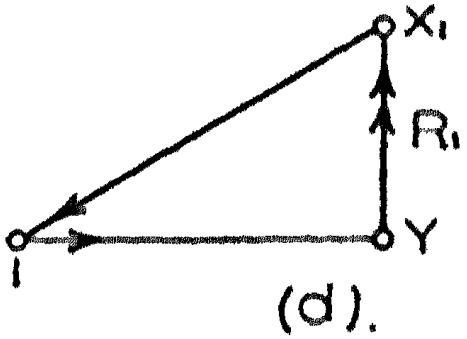

0'
10'
Scale of
R1
R2


P1
2
1

(e)


X1
1
Y
R1

(c)


X1
R1
Y
1

(d).

Joint L

toward the ends of the stress lin
and tension is indicated by arro
stress lines and away from the
stress is used a single arrow, an
double arrow is used to indicate
Fig. 27 is called a Maxwell diag
The notation used is known as Bo
resolution is the method most com
roof trusses and simple framed st

Algebraic Moments.—The
the fundamental equations of equ
In the truss in (a) Fig. 28 by tak
we have

To find the stresses in the mem
proceed as follows: Cut the truss by
replace the stresses in the member
These forces are equal to the stress
opposite in direction, and produce e

To obtain stress 4-x take cen
moments of external forces

$$4\text{-}x \times a + P_1 \times d$$

$$4\text{-}x = \frac{R_1 \times 2d -}{a}$$

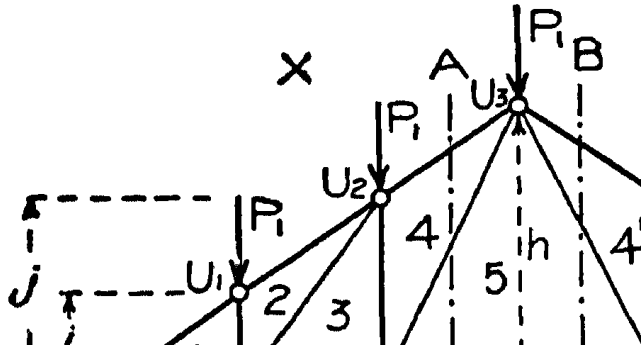
To obtain stress in 4-5 take ce
moments of external forces

$$4\text{-}5 \times b - 2P$$

taking moments about joint L
y-1 are found by taking mome
and the stresses in x-2 and x
joint L_1.

The method of algebraic
used for calculating the stress
and similar frameworks whic

Graphic Moments.—Th
truss may be found by mean



CHAPT

STRESSES IN SIM

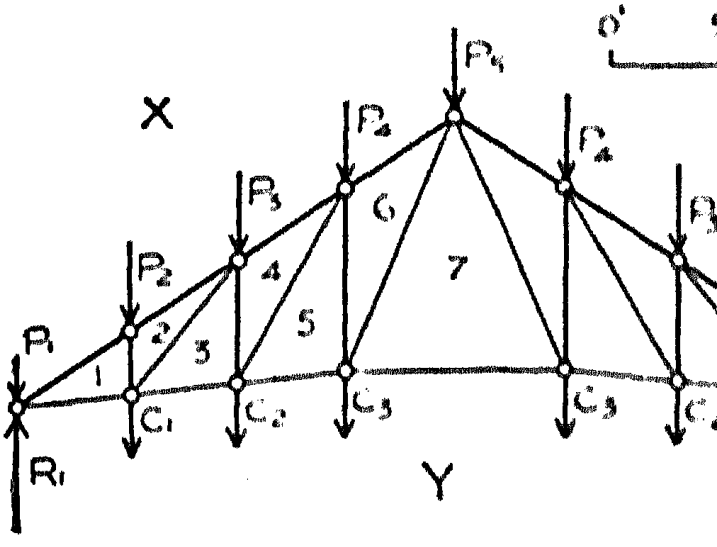
Loads.—The stresses in roof
(2) the snow load, (3) the wind lo
loads. The stresses due to dead,
will be discussed in this chapter in

Dead Load Stresses.—The
of the truss and roof covering an
the panel points of the upper c
trusses. If the purlins do not com

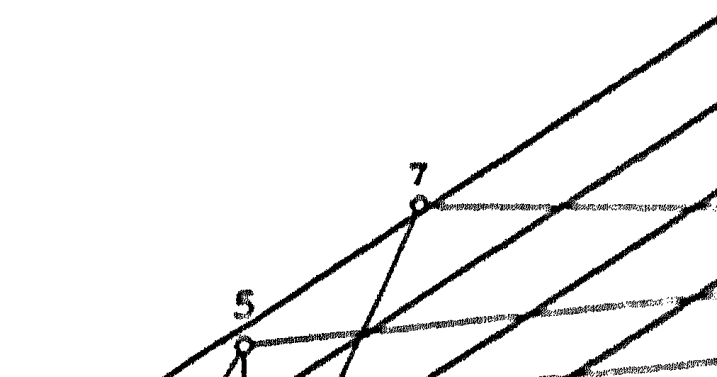
The loads are laid off, t
lated beginning at joint L_0, as
for the right half of the trus
loads are symmetrical as in
accuracy of the work and is
loads P_1 on the abutments h
and may be omitted in this s

In calculating the stresse
3-4, 4-5 and x-5 are unkno
determinate. The solution is
and 5-6, and replacing then
stresses in the members in t
and including stresses 6-x

Snow Load Stres



DEAD AND CEILING LOADS



and to the resultant of the ex
truss are equally free to move
reactions are equal; and where
on frictionless rollers (3) the
be vertical. Either case (1) or
culating wind load stresses in t
portal or framed bent. The v
independent of the condition of

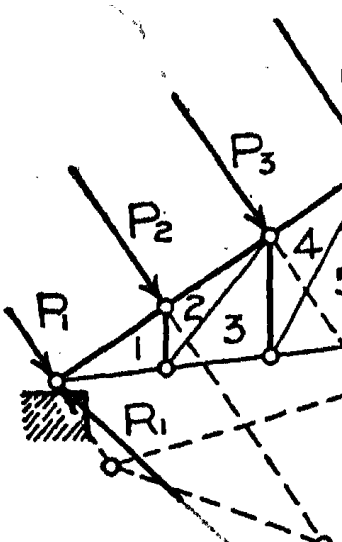
Wind Load Stresses: No

mal wind load, in a Fink truss
calculated by graphic resolution

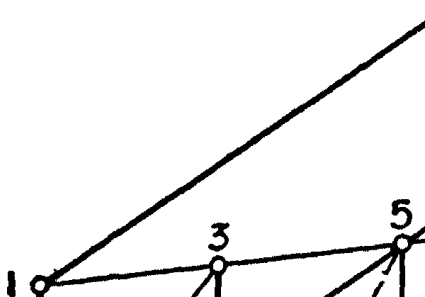
The stress diagram is constructed in
loads. Heavy lines in truss and st
and light lines indicate tension.

The ambiguity at joint P_a is rem
ber as in the case of the dead load s
there are no stresses in the dotted
of the truss. It is necessary to carr
truss, beginning at the left reaction
action. It will be seen that the load P
truss in this case.

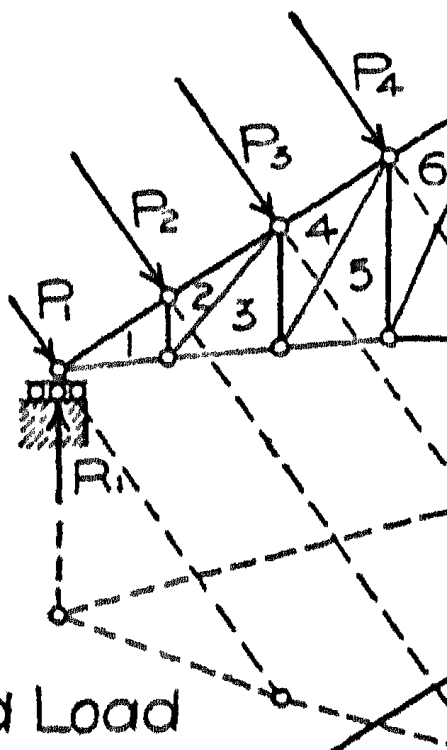
Wind Load Stresses: Roller are usually fixed at one end, and ar
end. The reaction at the roller end i



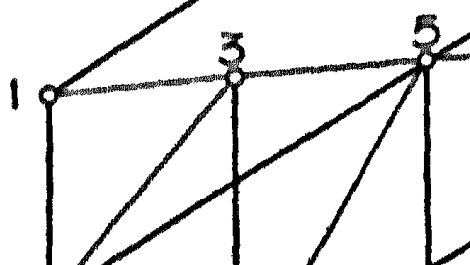
Wind Load
Rollers Leewa



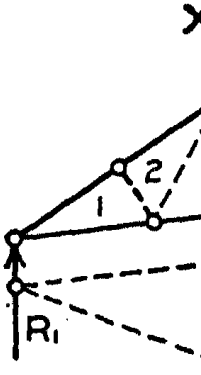
CONCENTRATED



Wind Load
Rollers Windward



31. The moving crane t
it is at the center of the
solving the problem.



CRANE L

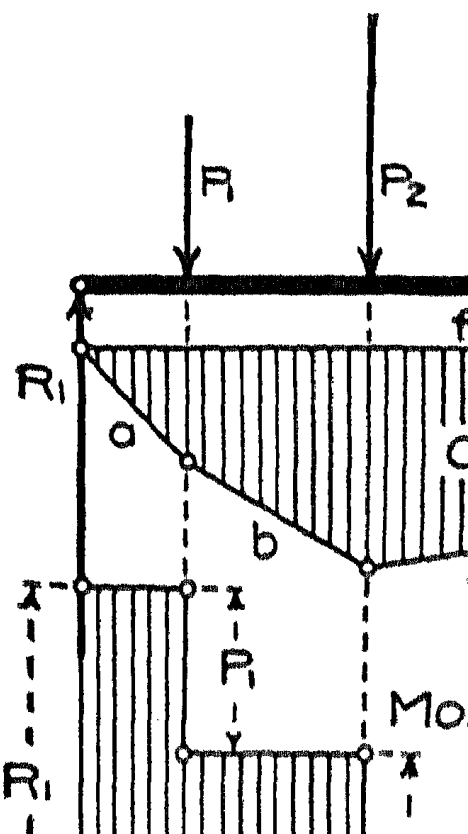
CHA

SIMP

Reactions.—The reactions
of the force and equilibrium poly
ond example let it be required t
beam shown in Fig. 36.

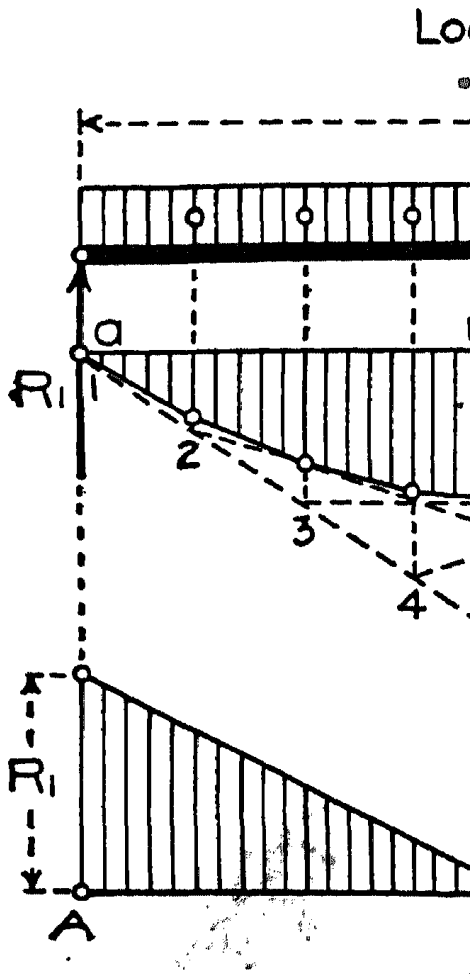
Moment and Shear

ing moment in the beam s
the force polygon (a) ar



R_1; between the loads P_1 and P_2 the
the loads P_2 and P_3 the shear equals R
P_3 and P_4 the shear equals $R_1 - P_1 -$
and the right reaction the shear equals
R_2. At load P_2 the shear changes fro
(c) is called a shear diagram. It
ordinate in the moment diagram co

The bending moment at any po
algebraic sum of the shear areas on ei
From this we see that the shear areas
This property of the shear diagram de
bending moment at any point in a simp
the shear between either point of su
This will be taken up again in the disc

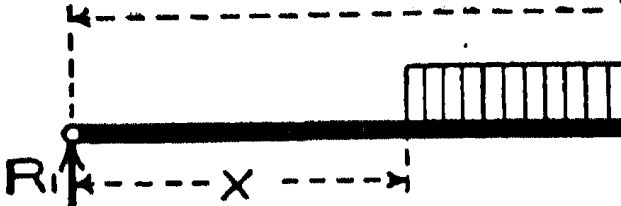


Shear

CHAPT

MOVING LOA

Uniform Moving Loads.—Le
a uniform load of p lbs. per lineal
the beam.



Differentiating (16) and placing derivative of M with respect to x equal to zero, we have after solving

$$x = o \tag{17}$$

Therefore the maximum moment at any point in a beam will occur when the beam is fully loaded.

The bending moment diagram for a beam loaded with a uniform moving load is constructed as in Fig. 38.

To find the position of the moving load for maximum shear at any point in a beam loaded with a moving uniform load, proceed as follows: The left reaction when the end of the moving load is at a distance x from the left support, will be

$$R_1 = \frac{(L - x)^2}{2L} p \tag{15}$$

and the shear at a point at a distance a from the left reaction will be

$$S = R_1 - (a - x)p = \frac{(L - x)^2}{2L} p - (a - x)p \tag{18}$$

which is the equation of a parabola.

By inspection it can be seen that S will be a maximum when $a = x$. The maximum shear at any point in a beam will therefore occur at the end of the uniform moving load, the beam being fully loaded to the right of the point as in (a) Fig. 39 for maximum positive shear, and fully loaded to the left of the point as in (b) Fig. 39 for maximum negative shear.

If the beam is assumed to be a cantilever beam fixed at A, and loaded with a stationary uniform load equal to p lbs. per lineal foot, and an equilibrium polygon be drawn with a force polygon having a pole distance equal to length of span, L, the parabola drawn through the points in the equilibrium polygon will be the maximum positive shear diagram, (a) Fig. 39. The ordinate at any point to this shear diagram will represent the maximum positive shear at the point to the same scale as the loads (for the application of this principal to bridge trusses see Fig. 50, Chapter X).

Concentrated Moving Loads.—Let a beam be loaded with concentrated moving loads at fixed distances apart as shown in Fig. 40.

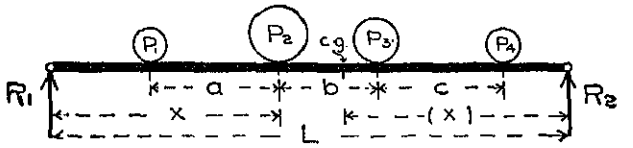


FIG. 40.

To find the position of the loads for maximum moment and the amount of the maximum moment, proceed as follows: The load P_2 will be considered first. Let x be the distance of the load P_2 from the left support when the loads produce a maximum moment under load P_2.

Taking moments about R_2 we have

$$R_1 = \frac{P_1(L-x+a)+P_2(L-x)+P_3(L-x-b)+P_4(L-x-b-c)}{L}$$

$$= \frac{(L-x)(P_1+P_2+P_3+P_4)+P_1 a-P_3 b-P_4(b+c)}{L} \qquad (19)$$

and the bending moment under load P_2 will be

$$M = R_1 x - P_1 a$$

$$= \frac{x(L-x)(P_1+P_2+P_3+P_4)+x(P_1 a-P_3 b-P_4(b+c))}{L} - P_1 a \qquad (20)$$

Differentiating (20) we have

$$\frac{dM}{dx} = \frac{(L-2x)(P_1+P_2+P_3+P_4)+P_1 a-P_3 b-P_4(b+c)}{L} = 0 \qquad (21)$$

and solving (21) for x we have

$$x = \frac{L}{2} + \frac{P_1 a - P_3 b - P_4(b+c)}{2(P_1+P_2+P_3+P_4)} \qquad (22)$$

Now $P_1 a - P_3 b - P_4(b+c)$, is the static moment of the loads about P_2 and

$$\frac{P_1 a - P_3 b - P_4(b+c)}{P_1 + P_2 + P_3 + P_4} = \text{distance from } P_2 \text{ to}$$

center of the gravity of all the loads.

Therefore, for a maximum moment under load P_2, it must be as far from one end as the center of gravity of all the loads is from the other end of the beam, Fig. 40.

The above criterion holds for all the loads on the beam. The only way to find which load produces the greatest maximum is to try each one, however, it is usually possible to determine by inspection which load will produce a maximum bending moment. For example the maximum moment in the beam in Fig. 40 will certainly come under the heavy load P_2. The above proof may be generalized without difficulty and the criterion above shown to be of general application.

For two equal loads $P = P$ at a fixed distance, a, apart as in the case of a traveling crane, Fig. 41, the maximum moment will occur under one of the loads when

$$x = \frac{L}{2} - \frac{a}{4}$$

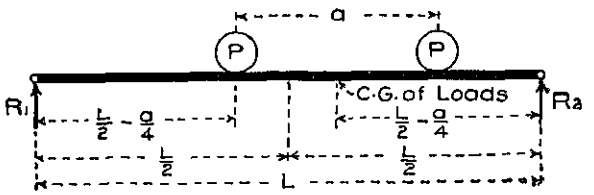


FIG. 41.

Taking moments about the right reaction we have

$$R_1 = \frac{P\left(L - \frac{a}{2}\right)}{L} \qquad (23)$$

and the maximum bending moment is

$$M = R_1\left(\frac{L}{2} - \frac{a}{4}\right)$$

$$= \frac{P\left(L - \frac{a}{2}\right)^2}{2L} \qquad (24)$$

There will be a maximum moment when either of the loads satisfies the above criterion, the bending moments being equal.

By equating the maximum moment above to the moment due to a single load at the center of the beam, it will be found that the above criterion holds only when

$$a < 0.586\ L$$

Where two unequal moving loads are at a fixed distance apart the greater maximum bending moment will always come under the heavier load.

The maximum end shear at the left support for a system of concentrated loads on a simple beam, as in Fig. 40, will occur when the left reaction, R_1, is a maximum. This will occur when one of the wheels is infinitely near the left abutment (usually said to be over the left abutment). The load which produces maximum end shear can be easily found by trial.

The maximum shear at any point in the beam will occur when one of the loads is over the point. The criterion for determining which load will cause a maximum shear at any point, x, in a beam will now be determined.

In Fig. 40, let the total load on the beam, $P_1 + P_2 + P_3 + P_4 = W$, and let x be the distance from the left support to the point at which we wish to determine the maximum shear.

When load P_1 is at the point, the shear will be equal to the left reaction, which is found by substituting $x + a$ for x in (19) to be

$$S_1 = R_1 = \frac{(L - x - a)\ W + P_1 a - P_3 b - P_4 (b + c)}{L}$$

and when P_2 is at the point the shear will be

$$S_2 = \frac{(L - x)\ W + P_1 a - P_3 b - P_4 (b + c)}{L} - P_1$$

Subtracting S_2 from S_1 we have

$$S_1 - S_2 = \frac{P_1 L - Wa}{L}$$

Now S_1 will be greater than S_2 if $P_1 L$ is greater than $W a$, or if

$$\frac{P_1 L}{a} > W$$

The criterion for maximum shear at any point therefore is as follows:

The maximum positive shear in any section of a beam occurs when the foremost load is at the section, provided W is not greater than $\frac{P_1 L}{a}$ If W is greater than $\frac{P_1 L}{a}$, the greatest shear will occur when some succeeding load is at the point.

Having determined the position of the moving loads for maximum moment and maximum shear, the amount of the moment and shear can be obtained as in the case of beams loaded with stationary loads.

CHAPTER X.

Stresses in Bridge Trusses.

Method of Loading.—The loads on highway bridges, and in y cases on railway bridges as well, are assumed to be concentrated e joints of the loaded chord, and if the panels of the truss are equal *oint loads* are equal. The assumption of joint loads simplifies the ion and gives values for the stresses that are on the safe side. Equal loads will be assumed in this discussion.

Algebraic Resolution.*—Let the Warren truss in Fig. 42 have loads applied at the joints as shown. From the fundamental tions for equilibrium for translation, reaction $R_1 = R_2 = 3\ W$.

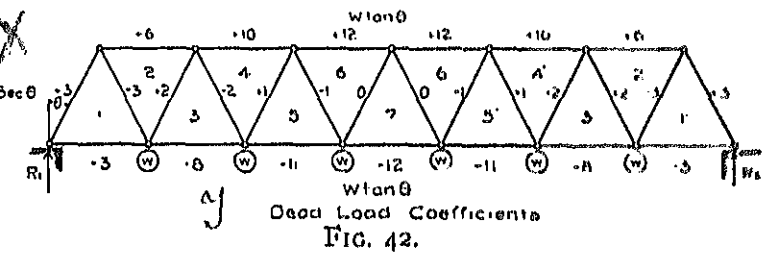


FIG. 42.

The stresses in the members are calculated as follows: Resolving he left reaction, stress in 1-x = + 3 W sec θ, and stress in 1-y = W tan θ. Resolving at first joint in upper chord, stress in 1-2 = W sec θ, and stress in 2-x = + 6 W tan θ. Resolving at second in lower chord, stress 2-3 = + 2 W sec θ, and stress 3-y = — tan θ. And in like manner the stresses in the remaining members found as shown. The coefficients shown in Fig. 42 for the chords to be multiplied by W tan θ; while those for the webs are to be iplied by W sec θ.

o called "Method of Sections."

It will be seen that the coefficients for the web stresses are equal to the shear in the respective panels. Having found the shears in the different panels of the truss, the remaining coefficients may be found by resolution. Pass a section through any panel and the algebraic sum of the coefficients will be equal to zero. Therefore, if two coefficients are known, the third may be found by addition.

Beginning with member 1-y, which is known and equals -3;

coefficient of 2-x $= -(-3-3) = +6$;
coefficient of 3-y $= -(+6+2) = -8$;
coefficient of 4-x $= -(-8-2) = +10$;
coefficient of 5-y $= -(+10+1) = -11$;
coefficient of 6-x $= -(-11-1) = +12$;
coefficient of 7-y $= -(+12+0) = -12$.

Loading for Maximum Stresses.—The effect of different positions of the loads on a Warren truss will now be investigated.

Let the truss in Fig. 43 be loaded with a single load P as shown.

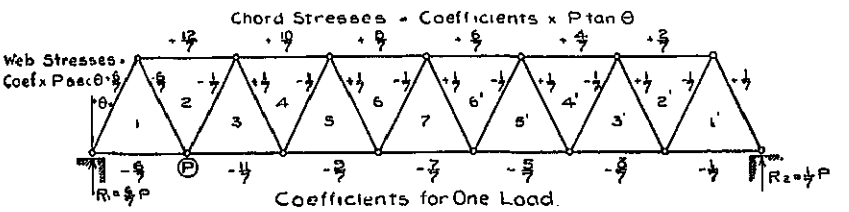


FIG. 43.

The left reaction, $R_1 = \frac{6}{7}P$, and the right reaction, $R_2 = \frac{P}{7}$. The stress in 1-y $= -\frac{6}{7}P \tan\theta$, and stress in 1-$x$ $= +\frac{6}{7}P \sec\theta$. The stress in 1-2 $= -\frac{6}{7}P \sec\theta$ and stress in 2-3 $= -\frac{1}{7}P \sec\theta$, etc. The remaining coefficients are found as in the case of dead loads by adding coefficients algebraically and changing the sign of the result.

In Fig. 44 the coefficients for a load applied at each joint in turn are shown for the different members; the coefficients for the load on left being given in the top line.

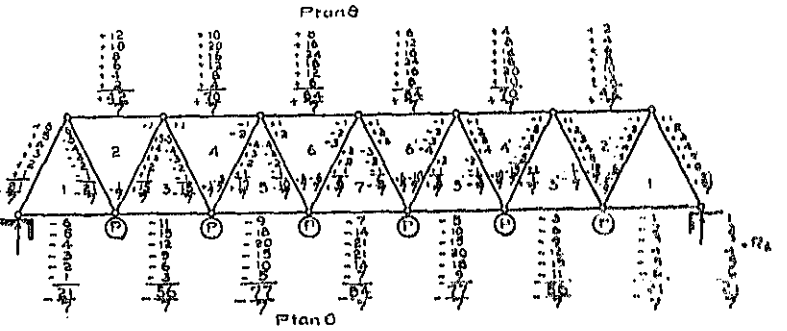


FIG. 44.

he following conclusions may be drawn from Fig. 44.

1) All loads produce a compressive stress in the top chord and le stress in the bottom chord.

2) All the loads on one side of a panel produce the same kind ss in the web members that are inclined in the same direction on de.

or maximum stresses in the chords, therefore, the truss should y loaded. For maximum stresses in the web members the longer rt into which the panel divides the truss should be fully loaded; or minimum stresses in the web members the shorter segment of ss should be fully loaded.

he conditions for maximum loading of a truss with equal joint re therefore seen to be essentially the same as the maximum load- a beam with a uniform live load.

resses in Warren Truss.—The coefficients for maximum and im stresses in a Warren truss due to live load are shown in Fig. 45. hese coefficients are seen to be the algebraic sum of the co- ts for the individual loads given in Fig. 44. The live load chord ents are the same as for dead load, and if found directly are in the same manner.

he maximum web coefficients may be found directly by taking off d at a time beginning at the left. The left reaction, which may nd by algebraic moments, will in each case be the coefficient of

the maximum stress in the panel to the left of the first load. A rule for finding the coefficient of left reaction for any loading is as follows: Multiply the number of loads on the truss by the number of loads plus unity, and divide the product by twice the number of panels in the truss and the result will be the coefficient of the left reaction.

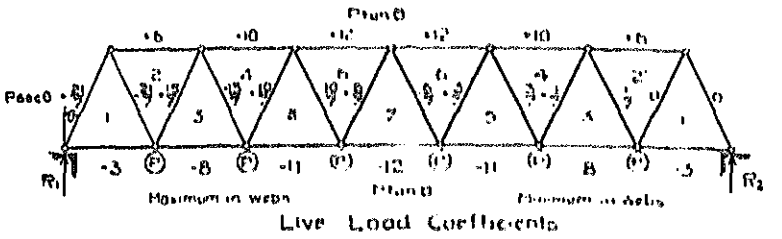


FIG. 45.

If the second differences of the maximum coefficients in the web members are calculated, they will be found to be constant, which shows that the coefficients are equal to the ordinates of a parabola.

1		
	2	
3		1
	3	
6		1
	4	
10		1
	5	
15		1
	6	
21		

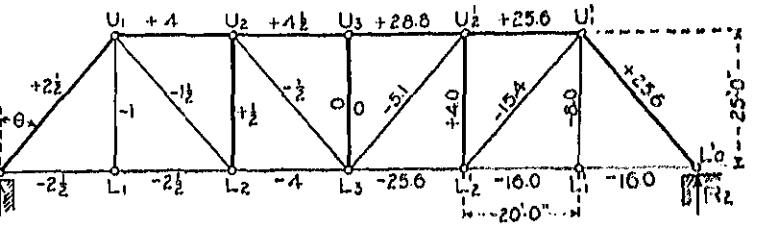
SECOND DIFFERENCES OF NUMERATORS OF WEB COEFFICIENTS.

This relation gives an easy method for checking up the maximum web coefficients, since the numerators of the coefficients are always the same beginning with unity in the first panel on the right and progressing in order 1, 3, 6, 10, etc.; the denominators always being the number of panels in the truss.

It should be noted that in the Warren truss the members meeting on the unloaded chord always have stresses equal in amount, but opposite in sign.

Stresses in Pratt Truss.— In the Pratt truss the diagonal members are tension members and counters (*see dotted members in (c) Fig. 46*) must be supplied where there is a reversal of stress. The coefficients for the dead and live load stresses in the Pratt truss shown in (a) and (b)

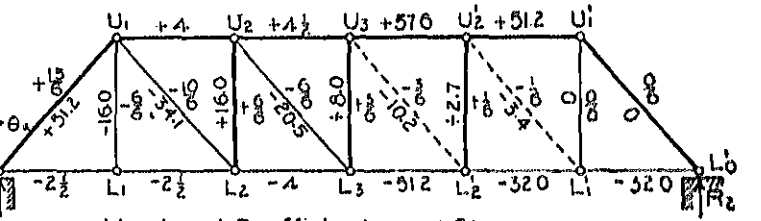
46, are found in the same manner as for a Warren truss. The ber $U_1 L_1$ acts as a hanger and carries only the load at its lower The stresses in the chords are found by multiplying the coeffi- by W tan θ, and in the inclined webs by multiplying the co- nts by W sec θ. The stresses in the posts are equal to the ver- components of the stresses in the inclined web members meeting on the unloaded chord.



Dead Load Coefficients
Dead Load = 8 Tons per Joint.

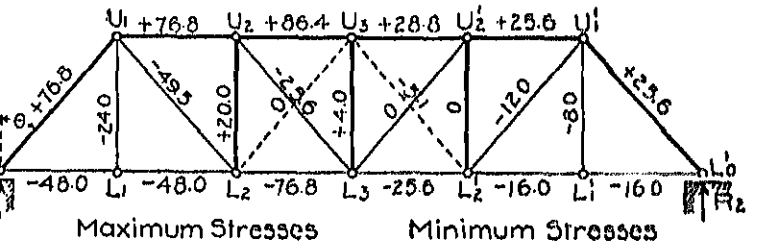
Dead Load Stresses
Sec θ = 1.28 · Tan θ = 0.80

(a)



Live Load Coefficients and Stresses
Live Load = 16 Tons per Joint. Sec θ = 1.28 · Tan θ = 0.80

(b)



Maximum Stresses Minimum Stresses

(c)

FIG. 46.

The maximum chord stresses shown on the left of (c) are equal to the sum of the live and dead load chord stresses. The minimum chord stresses shown on the right of (c) are equal to the dead load chord stresses.

The maximum and minimum web stresses are found by adding algebraically the stresses in the members due to dead and live loads.

Since the diagonal web members in a Pratt truss can take tension only, counters must be supplied as $U_3 L'_4$ in panel $L'_3 L_4$. The tensile stress in a counter in a panel of a Pratt truss is always equal to the compressive stress that would occur in the main diagonal web member in the panel if it were possible for it to take compression. Care must always be used to calculate the corresponding stresses in the vertical posts.

Graphic Resolution.—The stresses in a Warren truss due to dead loads are calculated by graphic resolution in Fig. 47. The solution is the same as for ceiling loads in a roof truss. The loads beginning with the first load on the left are laid off from the bottom upwards. The analysis of the solution is shown on the stress diagram and truss and needs no explanation.

From the stresses in the members it is seen (a) that web members meeting on the unloaded chord have stresses equal in amount but opposite in sign, and (b) that the lower chord stresses are the arithmetical means of the upper chord stresses on each side.

The live load chord stresses may be obtained from the stress diagram in Fig. 47 by changing the scale or by multiplying the dead load stresses by a constant.

The live load web stresses may be obtained by calculating the left reactions for the loading that gives a maximum shear in the panel (no loads occurring between the panel and the left reaction), and then constructing the stress diagram up to the member whose stress is required. In a truss with parallel chords it is only necessary to calculate the stress in the first web member for any given reaction since the shear is constant between the left reaction and the panel in question.

The live load web stresses may all be obtained from a single diagram as follows: With an assumed left reaction of, say, 100,000 lbs. onstruct a stress diagram on the assumption that the truss is a cantilever fixed at the right abutment and that there are no loads on the

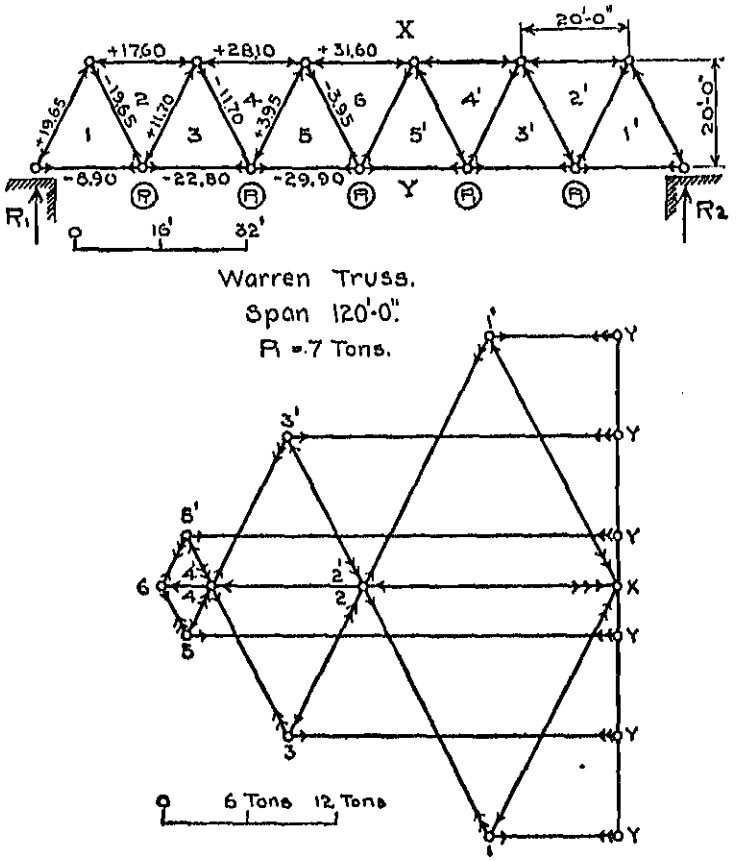


FIG. 47.

uss. Then the maximum stress in any web member will be equal to e stress scaled from the diagram, divided by 100,000, multiplied by e left reaction that produces the maximum stress. This method is a ery convenient one for finding the stresses in a truss with inclined ords.

Algebraic Moments.— The dead and live load stresses in a truss with inclined chords are calculated by algebraic moments in Fig. 48. The conditions for maximum loading are the same in this truss as in a truss with parallel chords, and are as follows: Maximum chord stresses occur when all loads are on; minimum chord stresses occur when no live load is on; maximum web stresses in main members occur when the longer segment of the truss is loaded; and minimum stresses in main members and maximum stresses in counters occur when the shorter segment of the truss is loaded. An apparent exception to the latter rule occurs in post $U_3 L_3$ which has a maximum stress when the truss is fully loaded with dead and live loads.

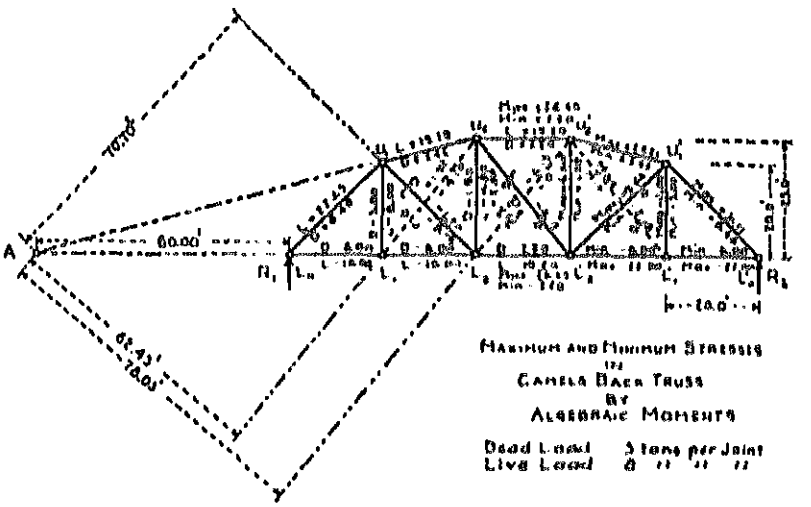


FIG. 48.

To find the stress in member $U_1 L_2$ take moments about point A, the intersection of the upper and lower chords produced. The dead load stress is then given by the equation

$$U_1 L_2 \times 70.7 + R_1 \times 60 - W \times 80 = 0$$
$$U_1 L_2 \times 70.7 = -6 \times 60 + 3 \times 80 = -120 \text{ foot-tons}$$
$$U_1 L_2 = -1.70 \text{ tons}$$

The maximum live load stress occurs when all loads are on except L_1, and

$$U_1 L_2 \times 70.7 + R_1 \times 60 = 0$$
$$U_1 L_2 \times 70.7 = -\tfrac{6}{5} P \times 60 = -576 \text{ foot-tons}$$
$$U_1 L_2 = -8.14 \text{ tons}$$

The maximum live load stress in counter $U_2 L_1$ occurs with a load at L_1, and is given by the equation

$$-U_2 L_1 \times 62.43 + R_1 \times 60 - P \times 80 = 0$$
$$U_2 L_1 \times 62.43 = \tfrac{4}{5} P \times 60 - 8 \times 80$$
$$U_2 L_1 = -4.10 \text{ tons}$$

The dead load stress in counter $U_2 L_1$ when main member $U_1 L_2$ is not acting will be

$$U_2 L_1 \times 62.43 = +120 \text{ foot-tons}$$
$$U_2 L_1 = +1.92 \text{ tons}$$

The maximum stress in $U_1 L_2$ is therefore $-1.70 - 8.14 = -9.84$ tons, and the minimum stress is zero. The maximum stress in counter $U_2 L_1$ is $+1.92 - 4.10 = -2.18$ tons, and the minimum stress is zero.

The stresses in the remaining members may be found in the same manner. To obtain stresses in upper chords $U_1 U_2$ and $U_2 U_2$, take moments about L_2 as a center; to obtain stress in lower chord $L_0 L_1$ take moments about U_1 as a center. The dead load and maximum live load stress in post $U_2 L_2$ is equal to the vertical component of the dead and live loads, respectively, in upper chord $U_1 U_2$. The stresses in $L_0 U_1$, $L_0 L_1$, $L_2 L'_2$, $U_2 U'_2$ and $U_2 L'_2$ are most easily found by algebraic resolution.

Graphic Moments.—The dead load stresses in the chords of a Warren truss are calculated by graphic moments in Fig. 49.

Bending Moment Polygon.—The upper chord stresses are given by the ordinates to the bending moment parabola direct, while the lower chord stresses are arithmetical means of the upper chord stresses

on each side, and are given by the ordinates to the chords of the parabola as shown in Fig. 49.

The parabola is constructed as follows: The mid-ordinate, $4j$, is made equal to the bending moment at the center of the truss divided by the depth; in this case the mid-ordinate is the stress in 6-x; if the number of panels in the truss were odd the mid-ordinate would not be equal to any chord stress. The parabola is then constructed as shown in Fig. 49. The live load chord stresses may be found from Fig. 49 by changing the scale or by multiplying the dead load chord stresses by a constant.

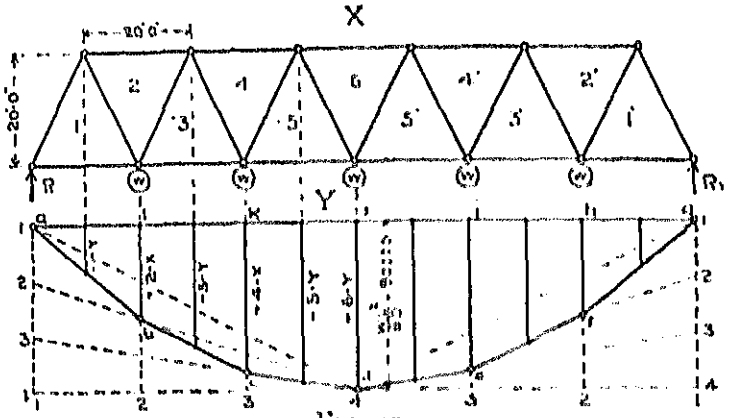


FIG. 49.

Shear Polygon.—In Chapter IX it was shown that the maximum shear in a beam at any point could be represented by the ordinate to a parabola at any point. The same principle holds for a bridge truss loaded with equal joint loads, as will now be proved.

In Fig. 50 assume that the simple Warren truss is fixed at the left end as shown, and that right reaction R_1 is not acting. Then with all joints fully loaded with a live load P, construct a force polygon as shown, with pole o and pole distance H = span L, and beginning at point a in the load line of the force polygon construct the equilibrium polygon $a\ g\ h$ for the cantilever truss.

Now the bending moment at the left support will be equal to

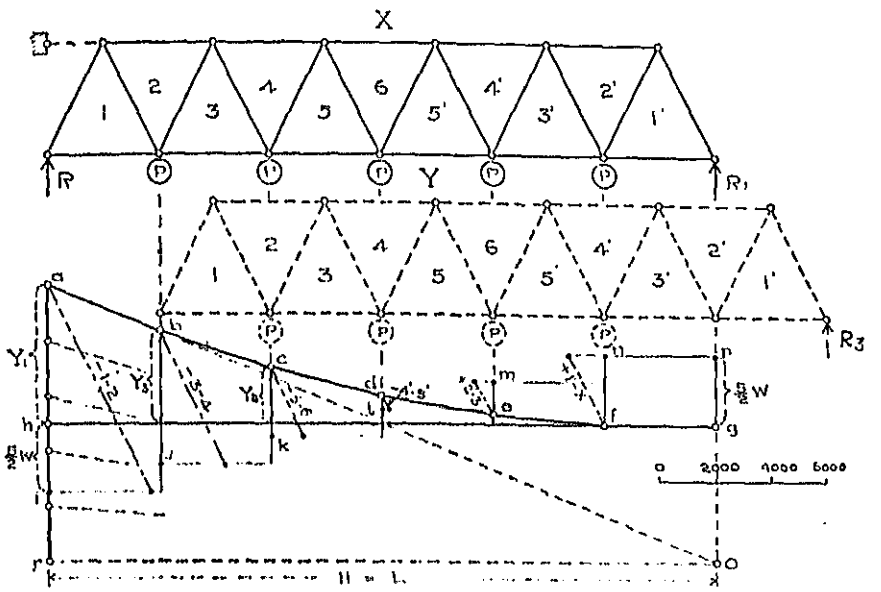


FIG. 50.

ordinate Y_1 multiplied by the pole distance H. But the truss is a simple truss and the moment of the right reaction will be equal to the moment at the left abutment and

$$Y_1 H = R_1 L$$

and since $H = L$

$$Y_1 L = R_1 L \qquad \text{and}$$

$$Y_1 = R_1$$

Now, with the loads remaining stationary, move the truss one panel to the right as shown by the dotted truss. With the same force polygon draw a new equilibrium polygon as above. This equilibrium polygon will be identical with a part of the first equilibrium polygon as shown. As above, the bending moment at left reaction is $Y_2 H = Y_2 L = R_2 L$, and $Y_2 = R_2$. In like manner Y_3 can be shown to be the right reaction with three loads on, etc. Since the bridge is symmetrical with reference to the center line, the ordinates to the shear polygon in Fig. 50, are equal to the maximum shears in the panel to the right of the ordinate as the load moves off the bridge to the right.

To draw the shear parabola direct, without the use of the force and equilibrium polygons proceed as follows: At a distance of a panel length to the left of the left abutment lay off to scale a load line equal to one-half the total load on the truss, divide this load line into as many parts as there are panels in the truss, and beginning at the top, which call 1, number the points of division of the load line 1, 2, 3, etc., as in Fig. 49. Drop vertical lines from the panel points and number them 1, 2, 3, etc., beginning with the load line, which will be numbered 1, the left reaction numbered 2, etc. Now connect the numbered points in the load line with the point *f*, which is under the first panel to the left of the right abutment; and the intersection of like numbered lines will give points on the shear parabola. It should be noted that the line *h g* is a secant to the parabola and not a tangent as might be expected.

The dead load shear is laid off positive downward in Fig. 50 to the same scale as the live load shears, and the maximum and minimum shears due to dead and live loads are added graphically. The stresses in the web members are calculated graphically in Fig. 50.

Wheel Loads.—The criteria for maximum moments and shears in bridge trusses loaded with wheel loads are as follows:

(1) *Maximum Moment* at any joint in a bridge loaded with wheel loads will occur when the average load on the left of the section is the same as the average load on the whole span.

(2) *Maximum Shear* in any panel in a bridge loaded with wheel loads will occur when the load on the panel is equal to the load on the bridge divided by the number of panels.

These criteria will be proved by means of the influence diagram in the following discussion.

For a more complete discussion of the subject see standard books on bridge design. For the calculation of stresses in simple trusses see problems in Appendix II.

INFLUENCE DIAGRAMS.—An influence diagram (commonly called an influence line) shows the variation of the effect of a moving load or a system of loads on a beam or truss. The difference between bending moment or shear diagrams and influence diagrams is that the bending moment and the shear diagram gives the moment and shear, respectively, at any point for a fixed system of loads, while an influence diagram gives the moment or shear, etc., at a fixed point for a moving system of loads. Influence diagrams are used principally for finding the position of moving loads that will produce maximum shears, moments, reactions, or stresses, although they may be used for calculating the quantities themselves. For convenience where a number of loads are considered the influence diagrams are drawn for a single unit load. The unit influence diagram may then be used for any load by multiplying by the given load. The unit influence diagram will be referred to in the following discussion.

Maximum Moment in a Truss or Beam.—Let P_1 in Fig. 50a, represent the summation of the moving loads to the left of the panel point 2′ and P_2 be the summation of the moving loads to the right.

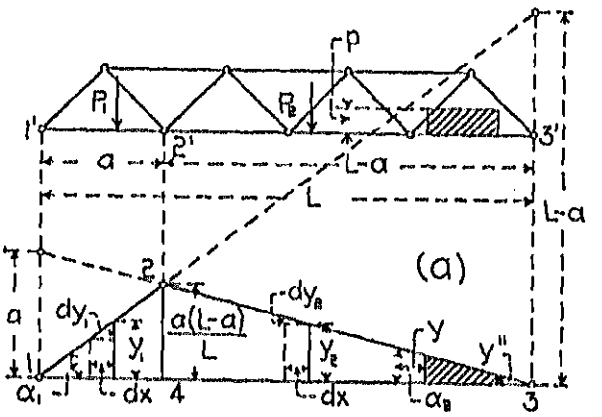


FIG. 50a. INFLUENCE DIAGRAM FOR MOMENTS.

The influence diagram for the point 2′ is constructed by calculating the bending moment at 2′ due to a unit load, $\frac{a(l-a)}{l}$ = ordinate

2–4, and drawing lines 1–2 and 2–3. The equation of the line 1–2 is

$$y = \frac{x(L - a)}{L}$$

and the equation of the line 2–3 is

$$y = \frac{a(L - x)}{L}$$

Now when $x = a$ the two lines have a common ordinate which is equal to $\frac{a(L-a)}{L}$. Also when $x = L$ the ordinate to 1–2 $= L - a$; while when $x = 0$, the ordinate to 2–3 is a, as is seen in Fig. 50a. This relation gives an easy method of constructing an influence diagram for moments for any point in a beam or truss.

Now in Fig. 50a the bending moment at x' due to the loads P_1 and P_2 is

$$M = P_1 y_1 + P_2 y_2 \qquad (a)$$

Now move the loads P_1 and P_2 a short distance to the left, the distance being assumed so small that the distribution of the loads will not be changed, and

$$M + dM = P_1 (y_1 - dy_1) + P_2 (y_2 + dy_2) \qquad (b)$$

Subtracting (a) from (b) and placing $dM = 0$, we have

$$dM = -P_1 dy_1 + P_2 dy_2 = 0$$

But $dy_1 = dx \tan a_1 = dx \frac{L - a}{L}$, and $dy_2 = dx \tan a_2 = dx \frac{a}{L}$, and

$$dM = -P_1 \frac{L - a}{L} dx + P_2 \frac{a}{L} dx = 0$$

from which $P_1 a - P_1 L + P_2 a = 0$, and $(P_1 + P_2)\, a = P_1 L$

Solving, we have

$$\frac{P_1}{a} = \frac{P_1 + P_2}{L} \qquad (c)$$

From (c) it follows *that the maximum bending moment at 2′ occurs when the average load on the left of the section is the same as the average load on the entire bridge.*

Uniform Loads.—In Fig. 50a, the bending moment at 2′ due to a uniform load $p\,d\,x$ will be $p\,y\,d\,x$ in (a). But $y\,d\,x$ is the area of the influence diagram under the uniform load, and the bending moment at 2′ due to a uniform load will be equal to the area of the influence diagram covered by the load, multiplied by the load per unit of length. For a uniform load, p, covering the entire span the bending moment at 2′ will be p times the area of the influence diagram 1–2–3. For a uniform load the bridge must be fully loaded to obtain maximum bending moment at any point. It will be seen that the general criterion for maximum moment is satisfied when the bridge is fully loaded with a uniform load.

Maximum Shear in a Truss.—Let P_1, P_2, and P_3 in Fig. 50b represent the loads on the left of the panel, on the panel, and on the right of the $(m+1)$st. panel, respectively. It is required to find the position of the loads for a maximum shear in the panel.

With a load unity at 2′ the shear in the panel is $-\frac{m}{n}$, and 1–2 is the influence shear line for loads to the left of the panel. With a load unity at 3′ the shear in the panel is $-\frac{n-m-1}{n}$, and 3–4 is the influence shear line for loads to the right of the panel. For a load on the panel the shear will vary from $-\frac{m}{n}$ at 2′ to $\frac{n-m-1}{n}$ at 3′, and the line 2–3 is the influence shear line for loads on the panel.

The influence diagram for the entire span is the polygon 1–2–3–4. It will be seen that the lines 1–2 and 3–4 are parallel, and are at a distance unity apart.

The total shear in the panel will then be

$$S = -P_1 y_1 + P_2 y_2 + P_3 y_3 \qquad (d)$$

Now move the loads a short distance to the left, the distance being assumed so small that the distribution of the loads will not be changed, and

$$S + dS = -P_1 (y_1 - dy_1) + P_2 (y_2 - dy_2) + P_3 (y_3 + dy_3) \quad (e)$$

Subtracting (d) from (e) and solving for a maximum

$$dS = P_1 dy_1 - P_2 dy_2 + P_3 dy_3 = 0$$

But

$$dy_1 = dx \tan \alpha_1 = dx \frac{1}{nl}$$

$$dy_2 = dx \tan \alpha_2 = dx \frac{n-1}{nl}$$

$$dy_3 = dx \tan \alpha_3 = dx \frac{1}{nl}$$

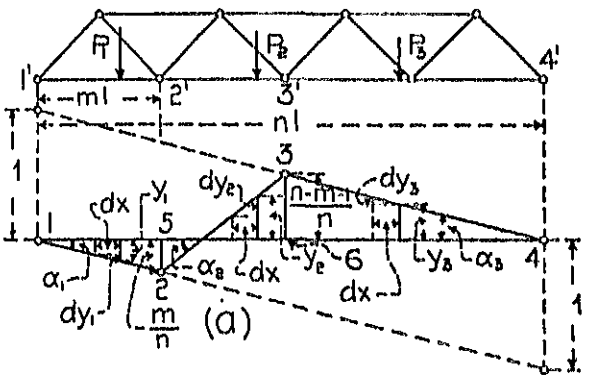


FIG. 50b. INFLUENCE DIAGRAM FOR SHEAR.

and substituting we have

$$dS = P_1 \frac{dx}{nl} - P_2 dx \frac{n-1}{nl} + P_3 \frac{dx}{nl} = 0$$

$$P_1 - P_2 (n-1) + P_3 = 0$$

and

$$P_1 + P_2 + P_3 = P_2 n$$

and

$$P_2 = \frac{P_1 + P_2 + P_3}{n} \qquad (f)$$

om (f) it follows that *the maximum shear in the panel will occur en the load on the panel is equal to the load on the bridge divided the number of panels in the bridge.*

Uniform Loads.—From Fig. 50b, it will be seen that for a uni- m load the maximum shear in the panel will occur when the uniform l extends from the right abutment to that point in the panel where line 2–3 passes through the line 1–4 (where the shear changes sign). r a minimum shear in the panel (maximum shear of the opposite n) the load should extend from the left abutment to the point in panel where the shear changes sign. For equal joint loads, load longer segment for a maximum shear in the panel, and load the rter segment for a minimum shear in the panel.

Maximum Floor Beam Reaction.—It is required to find the ximum load on the floor beam at 2′ in (a) Fig. 50c for the loads ried by the floor stringers in the panels 1′–2′ and 2′–3′.

In (a) the diagram 1–2–3 is the influence diagram for the shears 2′ due to a load unity at any point in either panel. In (b) the dia- m 1–2–3 is the influence diagram for bending moment at 2′ for a t load at any point in the beam. Now diagram in (a) differs from gram in (b) only in the value of the ordinate 2–4. It will be seen t the reaction at 2′ in (a) may be obtained from the diagram (b) for any system of loads if the ordinates are multiplied by $\frac{d_1 + d_2}{d_2}$. We can therefore use diagram (b) for obtaining the ximum floor beam reaction if we multiply all ordinates by $\frac{d_1 + d_2}{d_1 d_2}$.

To obtain the maximum floor beam reaction, therefore, take a ple beam equal to the sum of the two panel lengths, and find the ximum bending moment at a point in the beam corresponding to panel point. This maximum moment multiplied by $\frac{d_1 + d_2}{d_1 d_2}$ *will be maximum floor beam reaction.* If the two panels are equal in

length the maximum bending moment at the center of the beam multiplied by $\frac{2}{d}$, where d is the panel length, will give the maximum floor beam reaction.

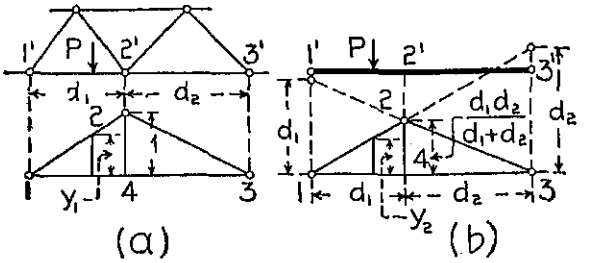


FIG. 50c. INFLUENCE DIAGRAM FOR MAXIMUM FLOOR BEAM REACTION.

CHAPTER XI.

Stresses in a Transverse Bent.

Dead and Snow Load Stresses.—The stresses due to the dead load in the trusses of a transverse bent are the same as if the trusses were supported on solid walls. The stresses in the supporting columns are due to the dead load of the roof and the part of the side walls supported by the columns, and are direct compressive stresses if the columns are not fixed at the top. If the columns are fixed at the top the deflection of the truss will cause bending stress in the columns. The dead load produces no stress in the knee braces of a bent of the type shown in Fig. 1 except that due to deflection of the truss, which may usually be omitted The stresses may be computed by algebraic or graphic methods.

The stresses due to snow load are found in the same way as the dead load stresses. In localities having a heavy fall of snow the freezing and thawing often cause icicles to form on the eaves of sufficient weight to tear off the cornice, unless particular care has been exercised in the design of this detail.

Wind Load Stresses.—The analysis of the stresses in a bent due to wind loads is similar to the analysis of the stresses in the portal of a bridge. The external wind force is taken (1) as horizontal or (2) as normal to all surfaces. The first is the more common assumption, although the second is more nearly correct. For a comparison of the stresses in a bent due to the wind acting horizontal and normal, see Figs. 54, 55, 56 and 57, and Table V. In the discussion which immediately follows, *the wind force will be assumed to act horizontally.*

The magnitude of the wind stresses in the trusses, knee braces and

columns will depend (a) upon whether the bases of the columns are fixed or free to turn, (b) upon whether the columns are rigidly fixed to the truss at the top, and (c) upon the knee brace and truss connections. Of the numerous assumptions that might be made, only two, the most probable, will be considered, viz.: (I) columns pin connected (free to turn) at the base and top, and (II) columns fixed at the base and pin connected at the top.

Columns in mill buildings are usually fixed by means of heavy bases and anchor bolts. Where the columns support heavy loads the dead load stress in the columns will assist somewhat in fixing them. Where the dead load stress plus algebraically the vertical component of the wind stress in the column, multiplied by one half the width of the base of the column parallel to the direction of the wind, is greater than the bending moment developed at the base of the leeward *column when the columns are considered as fixed, the columns* will be fixed without anchor bolts (see Chapter XII, Fig. 61). In any case the resultant moment is all that will be taken by the anchor bolts. The dead load stresses in mill buildings are seldom sufficient to give material assistance in fixing the columns. Unless care is used in anchoring columns it is best to design mill buildings for columns hinged at the base.

The general problem of stresses in a transverse bent for Case I and Case II, in which the stresses and forces are determined by algebraic methods, will now be considered. The application of the general problem will be further explained by the graphic solution of a particular problem.

ALGEBRAIC CALCULATION OF STRESSES: Case I. Columns Free to Turn at Base and Top. In Fig. 51, $H = H' = \frac{W'}{2}$ = horizontal reaction at the base of the column due to external wind force, W'.

$V' = -V'\frac{W'l}{2s}$ = vertical reaction at base of column due to the wind force, W'.

The wind produces bending in the columns, and also the direct

stresses V and V^1. Maximum bending occurs at the foot of the knee brace and is equal to $(H - W_7)\ d$ on the windward side, and $H^1\ d$ on the leeward side. These bending moments are the same as the bending moments in a simple beam supported at both ends and loaded with a concentrated load at the point of maximum moment. Since the max-

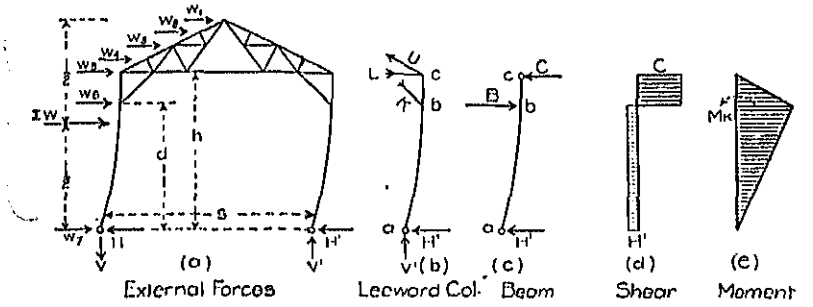


FIG. 51.

imum moment occurs at the foot of the knee brace in the leeward column, we will consider only that side. We will assume that the leeward column (b), Fig. 51, acts as a simple beam with reactions H^1 and C and a concentrated load B, as in (c). The reaction C and load B will now be calculated.

From the fundamental equation of equilibrium, summation horizontal forces equal zero, we have

$$B = H^1 + C \tag{25}$$

Taking moments about b, we have

$$C\ (h - d) = H^1\ d$$

$$C = \frac{H^1\ d}{h - d} \tag{26}$$

The stresses K, U and L can be computed by means of the following formulas:

$$K = B \text{ cosecant } m \tag{27}$$

where m = angle knee brace makes with column;

$$U = (V^1 - K \cos m) \text{ cosecant } n \tag{28}$$

where n = angle of pitch of roof; and

$$L = C - U \cos u \qquad (29)$$

In calculating the corresponding stresses on the windward side, the wind components acting at the points (a), (b) and (c) must be subtracted from H, B and C.

The shear in the leeward column is equal to H^1 below and C above the foot of the knee brace, (d) Fig. 51.

The moment in the column is shown in (e), Fig. 51, and is a maximum at the foot of the knee brace and is, $M = H^1 d$.

The maximum fibre stress due to wind moment and direct loading in the columns will occur at the foot of the knee brace in the leeward column, and will be compression on the inside and tension on the outside fibres, and is given by the formula*

$$f_2 + f_1 = \frac{P}{A} + \frac{M y}{I \pm \frac{P h^2}{10 E}} \qquad (30)$$

where f_1 = maximum fibre stress due to flexure;

f_2 = fibre stress due to direct load P;

A = area of cross-section of column in square inches;

M = bending moment in inch-pounds = $H^1 d$;

y = distance from neutral axis to extreme fibre of column in inches;

I = Moment of Inertia of column about an axis at right angles to the direction of the wind;

P = direct compression in the column in pounds;

h = length of the column in inches;

E = the modulus of elasticity of steel = 28,000,000;

$\frac{P h^2}{10 E}$ is minus when P is compression and plus when P is tension.

The maximum compressive wind stress is added to the direct dead and minimum snow load compression and governs the design of the column.

---

*This formula was first deduced by Prof. J. B. Johnson. For deduction of the formula see Chapter XV, or "Modern Framed Structures" by Johnson, Bryan and Turneaure.

Having the stresses K, U, and L, the remaining stresses in the truss can be obtained by ordinary algebraic or graphic methods.

For a simple graphic solution of the stresses in a bent for Case I, in which these stresses are computed graphically, see Fig. 54 for wind horizontal, and Fig. 56 for wind normal to all surfaces.

Case II. Columns Fixed at the Base.—With columns fixed at the base the columns may be (1) hinged at the top, or (2) rigidly fixed to the truss.

(1) *Columns fixed at the base and hinged at the top.*—It will be further assumed that the deflections at the foot of the knee brace and the top of the column, Fig. 52, are equal.

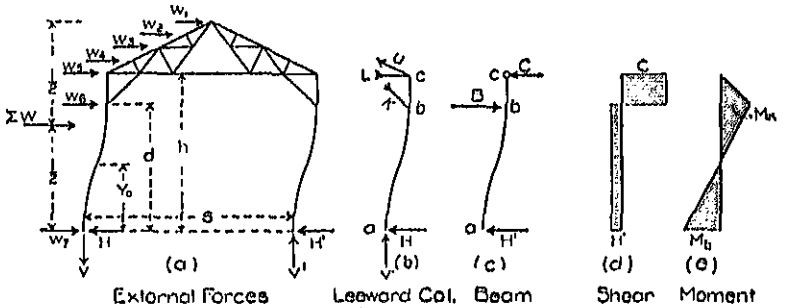


FIG. 52.

In Fig. 52 we have as in Case I

$$H = H^1 = \frac{W}{2}$$

V and V^1 are not as easily found as in Case I, but will be calculated presently.

The leeward column will be considered and will have horizontal external forces acting on it as shown in (c) Fig. 52. For convenience we will consider the leeward column as a beam fixed at a and acted upon by the horizontal forces B and C as shown in (c) Fig. 52, the deflection of the points b and c being equal by hypothesis.

From the fundamental condition of equilibrium, summation horizontal forces equal zero, we have

$$B = H^1 + C \qquad (31)$$

To obtain B and C a second equation is necessary.

From the theory of flexure we have for the bending moment in the column at any point y, where the origin is taken at the base of the column, when $y \leq d$

$$M = EI\frac{d^2x}{dy^2} = B(d-y) - C(h-y) \tag{32}$$

Integrating (32) between the limits $y = o$ and $y = d$, we have

$$EI\frac{dx}{dy} = \left[Bdy - \frac{By^2}{2} - Chy + \frac{Cy^2}{2} + F\right]_{y=o}^{y=d}$$

$$= -C\left(hd - \frac{d^2}{2}\right) + \frac{Bd^2}{2} \tag{33}$$

Now (33) equals EI times the angular change in the direction of the neutral axis of the column from $y = o$ to $y = d$.

When $y \geq d$, we have

$$M = EI\frac{d^2x}{dy^2} = -C(h-y) \tag{34}$$

Integrating (34) we have

$$EI\frac{dx}{dy} = -Chy + \frac{Cy^2}{2} + F_2 \tag{35}$$

Now (35) equals EI times the change in direction of the neutral axis of the column at any point from $y = d$ to $y = h$.

To determine the constant F_2 in (35) we have the condition that the angle at $y = d$ must be the same whether determined from equation (33) or equation (35). Equating (33) and (35) and making $y = d$, we have

$$F_2 = \frac{Bd^2}{2} \tag{36}$$

Substituting this value of F_2 in (35) we have

$$EI\frac{dx}{dy} = -Chy + \frac{Cy^2}{2} + \frac{Bd^2}{2} \tag{37}$$

Integrating (37) between the limits $y = d$ and $y = h$, we have

$$EIx = \left[-\frac{Chy^2}{2} + \frac{Cy^3}{6} + \frac{Bd^2y}{2}\right]_{y=d}^{y=h}$$

$$= C\left(\frac{hd^2}{2} - \frac{d^3}{6} - \frac{h^3}{3}\right) + B\left(\frac{d^2h}{2} - \frac{d^3}{2}\right) \tag{38}$$

Now (38) equals $E\ I$ times the deflection of the column from $y = d$ to $y = h$, which equals zero by hypothesis.

Solving (38) we have

$$\frac{C}{B} = \frac{3\,d^2\,h - 3\,d^3}{3\,h\,d^2 + d^3 + 2\,h^3}$$

$$= \frac{3\,d^2}{2\,h^2 + 2\,d\,h - d^2} \tag{39}$$

In a beam fixed at one end there is a point of inflection at some point, between $y = o$ and $y = d$, where the bending moment equals zero. Now if y_0 equals the value of y for the point of inflection, we have from (32)

$$B\ (d - y_0) = C\ (h - y_0) \qquad \text{and}$$

$$\frac{C}{B} = \frac{d - y_0}{h - y_0} \tag{40}$$

Equating the second members of equations (39) and (40) and solving for y_0, we have

$$y_0 = \frac{d}{2}\left(\frac{d + 2\,h}{2\,d + h}\right) \tag{41}$$

To find the relations between y_0 and d, we will substitute h in terms of d in (41) and solve for y_0.

For $\quad d = \frac{h}{2}, \qquad y_0 = \frac{5}{8}\,d,$

$$d = \frac{2}{3}\,h, \qquad y_0 = \frac{4}{7}\,d$$

$$d = h, \qquad y_0 = \frac{1}{2}\,d$$

Solving (31) and (39) for C, we have

$$C = \frac{H}{2}\,\frac{8\,d^2}{(h - d)\,(h + 2\,d)} \tag{42}$$

To find the moment M_b at the base of the leeward column, we have from (32)

$$M_b = B\,d - C\,h$$

Substituting the value of B given in (31) we have

$$M_b = H^1 d + C d - C h \tag{32a}$$

Eliminating h and d by means of (41) and (42) we have finally

$$M_k = H^1 y_0 \tag{43}$$

In like manner it can be shown that the moment at the base of the windward column is

$$M_{b_1} = H y_0 - \frac{w d^2}{8} \tag{43a}$$

where w equals the wind load per foot of height.

To find V we will take moments about the leeward column. The moments M_b and M_{b_1} at the bases of the columns respectively, resist overturning and we have

$$V = - V^1 = \frac{1}{s}\left[\frac{w l^2}{2} - M_b - M_{b_1}\right]$$

and since $$H = \frac{w l}{2}$$

$$V = \frac{1}{s}\left[H l - 2 H y_0 + \frac{w d^2}{8}\right] \tag{44}$$

Now if $\frac{d}{2}$ is taken equal to y_0, we have after transposing

$$V = - V^1 = \frac{1}{s}[2 H - w y_0]\left[\frac{l - y_0}{2}\right] \tag{45}$$

It will be seen that (45) is the same value of V and V^1 that we would obtain if the bent were hinged at the point of contra-flexure.

From (43) and (45) it will be seen that we can consider the columns as hinged at the point of contra-flexure and solve the problem as in Case I, taking into account the wind above the point of contra-flexure only. The maximum shear in the column is shown in (d) Fig. 52.

The maximum positive moment occurs at the foot of the leeward knee brace and is $M_k = H (d - y_0)$; the maximum negative moment occurs at the base of the leeward column and is equal to $M_b = H y_0$.

The maximum fibre stress occurs at the foot of the knee brace, and is given by the formula

$$f_2 + f_1 = \frac{P}{A} + \frac{M y}{I \pm \frac{I' h^2}{10 E}} \qquad (30a)$$

The nomenclature being the same as for (30) except h, which is the distance in inches from the point of contra-flexure to the top of the column.

(2) *Columns fixed at the base and top.*—In this case it can be seen by inspection that the point of inflection is at a point $y_0 = \frac{d}{2}$ and we have for this case

$$B = H^1 + C \qquad (31a)$$

$$M_b = \frac{H^1 d}{2} = M_k \qquad (32a)$$

$$C = \frac{H^1 d}{2(h - d)} \qquad (42a)$$

It is difficult to realize the exact conditions in either (1) or (2), in Case II, and it is probable that when an attempt is made to fix columns at the base, the actual conditions lie some place between (1) and (2). It would therefore seem reasonable to assume the minimum value, $y_0 = \frac{d}{2}$ as the best value to use in practice. This assumption is commonly made and will be made in the problems which follow.

Having the external forces H^1, B, C and V^1 the stresses K, U and L are computed by formulas (27), (28) and (29). The remaining stresses in the truss can then be computed by the ordinary algebraic or graphic methods.

For a simple graphic solution of this problem, where the external forces B and C are not computed, see Fig. 55 and Fig. 57.

Maximum Stresses.—It is not probable that the maximum snow and wind loads will ever come on the building at the same time, and it is therefore not necessary to design the structure for the sum of the maximum stresses due to dead load, snow load and wind load. A

common method is to combine the dead load stresses with the snow or the wind load stresses that will produce maximum stresses in the members. It is, however, the practice of the author to consider that a heavy sleet may be on the roof at the time of a heavy wind, and to design the structure for the maximum stresses caused by dead and snow load; dead load, minimum snow load and wind load; or dead load and wind load. It should be noted that the maximum reversals occur when the dead and wind load are acting. For a comparison of the stresses due to the different combinations see Table VI.

A common method of computing the stresses in a truss of the Fink type for small steel frame mill buildings is to use an equivalent uniform vertical dead load; the knee braces and the members affected directly by the knee braces being designed according to the judgment of the engineer. This method is satisfactory and expeditious when used by an experienced man, but like other short cuts is dangerous when used by the inexperienced. For a comparison of the stresses in a 60-foot Fink truss by the exact and the approximate method above, see Table VI.

Stresses in End Framing.—The external wind force on an end bent will be one-half what it would be on an intermediate transverse bent, and the shear in the columns may be taken as equal to the total external wind force divided by the number of columns in the braced panels. The stresses in the diagonal rods in the end framing, as in Fig. 1, will then be equal to the external wind force *H*, divided by the number of braced panels, multiplied by the secant of the angle the diagonal rod makes with a vertical line (For analysis of Portal Bracing see Chapter XII).

Bracing in the Upper Chord and Sides.—The intensity of the wind pressure is taken the same on the ends as on the sides, and the wind loads are applied at the bracing connection points along the end rafters and the corner columns. The shear transferred by each braced panel is equal to the total shear divided by the number of braced panels. The stresses in the diagonals in each braced panel are com-

puted by applying wind loads at the points above referred to, the wind loads being equal to the total wind loads divided by the number of panels. The stresses are computed as in a cantilever truss. The bracing in the plane of the lower chord is designed to prevent undue deflection of the end columns and to brace the lower chords of the trusses. All wind braces should be designed for, say, 5,000 pounds initial stress in each member, and the struts and connections should be proportioned to take the resulting stresses.

It should be noted that a mill building can be braced so as to be rigid without knee braces if the bracing be made sufficiently strong.

GRAPHIC CALCULATION OF STRESSES.—Data.—To illustrate the method of calculating the stresses in a transverse bent by graphic methods, the following data for a transformer building similar to one designed by the author will be taken.

The building will consist of a rigid steel frame covered with corrugated steel and will have the following dimensions: Length of building, 80′ 0″; width of building, 60′ 0″; height of columns, 20′ 0″; pitch of truss, ¼ (6″ in 12″); total height of building, 35′ 0″; the trusses will be spaced 16′ 0″ center to center. The trusses will be riveted Fink trusses. Purlins will be placed at the panel points of the trusses and will be spaced for a normal roof load of 30 lbs. per square foot. The roof covering will consist of No. 20 corrugated steel with 2½-inch corrugations, laid with 6-inch end laps and two corrugations side lap, with anti-condensation lining (see Chapter XVIII). The side covering will consist of an outside covering of No. 22 corrugated steel with 2½-inch corrugations, laid with 4-inch end laps and one corrugation side lap; and an inside lining of No. 24 corrugated steel with 1¼-inch corrugations, laid with 4-inch end laps and one corrugation side lap. For additional warmth two layers of tar paper will be put inside of the lining. Three 36-inch Star ventilators placed on the ridge of the roof will be used for ventilation. The general arrangement of the framing and bracing will be as in Fig. 1 and Fig. 81.

The approximate weight of the roof per square foot of horizontal projection will be as follows:

Trusses	3.6	lbs. per sq. ft.
Purlins and Bracing	3.0	" " " "
Corrugated Steel	2.4	" " " "
Roof Lining	1.0	" " " "
Total	10.0	" " " "

The maximum snow load will be taken at 20 pounds, and the minimum snow load at 10 pounds per square foot of horizontal projection of roof.

The wind load will be taken at 20 pounds per square foot on a vertical projection for the sides and ends of the building, 20 pounds per square foot on a vertical surface when the wind is considered as acting horizontally on the vertical projection of the roof, and 30 pounds per square foot on a vertical surface when the wind is considered as acting normal to the roof.

The stresses in an intermediate transverse bent will be calculated for the following:

CASE 1. Permanent dead and snow loads.

CASE 2. A horizontal wind load of 20 pounds per square foot on the sides and vertical projections of the roof, with the columns hinged at the base.

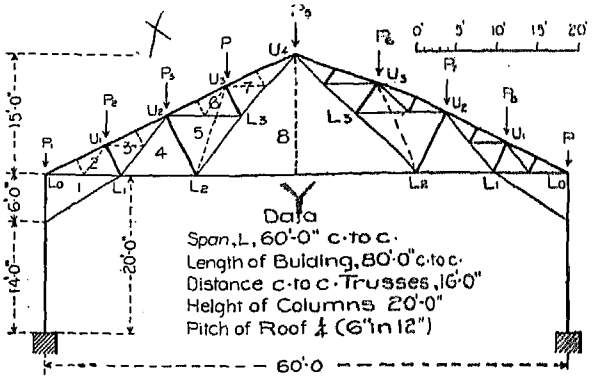
CASE 3. Same wind load as in Case 2, with columns fixed at the base.

CASE 4. A horizontal wind load of 20 pounds per square foot on the sides, and the normal component of a horizontal wind load of 30 pounds per square foot on the roof, with columns hinged at the base.

CASE 5. Same wind load as in Case 4, with columns fixed at the base.

Case 1. Permanent Dead and Snow Load Stresses.—On account of the limited size of the stress diagram the secondary members have been omitted and the loads applied as shown in Fig. 53. The

loads producing stresses in the truss are laid off to the prescribed scale, x_1-y being the left, and y-x_8 the right reaction. The stresses are calculated as follows: Beginning with the left reaction, x_1-y, draw lines through x_1 and y, parallel to the upper and lower chords of the truss, respectively, and the line x_1-2 will represent the compressive stress in the member x_1-2 and y-2 will represent the tensile stress in the member y-2 to the scale of the stress diagram.



CASE 1

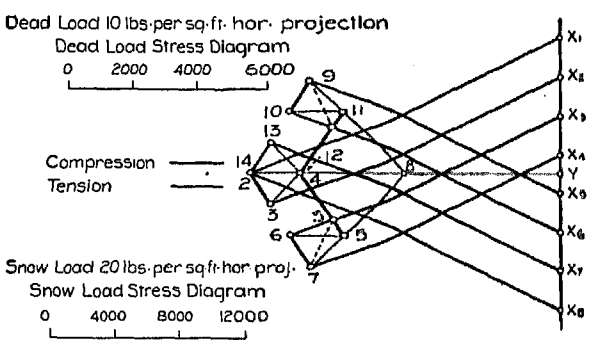


FIG. 53. DEAD AND SNOW LOAD STRESS DIAGRAM.

Calculate the stresses in the remaining members in like manner, being careful to take the members in order around a joint in completing any polygon. The indeterminate case at the joint U_3, can b

solved by calculating the stress in 5-6 and substituting it in the diagram, or by substituting an auxiliary member as shown. Compression and tension in the truss and stress diagram in Fig. 53 are indicated by heavy and light lines respectively.

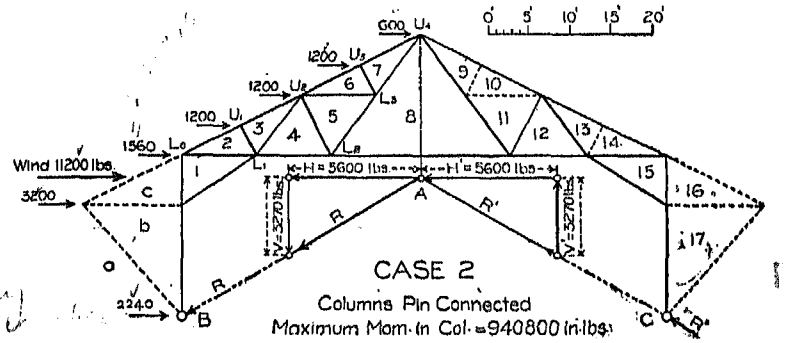
The stress in each column is equal to one-half the sum of the vertical loads, plus the load carried directly by the column.

Case 2. Wind Load Stresses: Wind Horizontal; Columns Hinged.—The wind will be considered as acting at the joints, as shown in Fig. 54. Replace the columns with trusses as indicated by the dotted lines. This makes the bent a two-hinged arch (see Chapter XIV), and the stresses will be statically determinate as soon as the horizontal reactions H and H^1 at the bases of the columns, have been determined. The usual assumption in mill buildings and portals of bridges is that $H = H^1 = \frac{W}{2}$ where W = the horizontal component of the external wind force (see Chapter XII). To calculate I' and I^1 graphically, produce the line of resultant wind until it intersects a vertical line through the center of the truss, and connect the intersection A with the bases of the columns B and C. From A lay off $II = II^1 = \frac{W'}{2}$, as shown in Fig. 54, and complete the triangles by drawing vertical lines through the ends of these lines. The vertical closing lines will be $I' = I^1$, as shown in Fig. 54.

The stresses are calculated as follows: Beginning with the foot of the column B, lay off the dotted line A-$B = R$. At B, lay off the load a-B = 2240 lbs.; through a draw a line parallel to auxiliary truss member a-b, and through A draw a line parallel to the column b-A, completing the polygon A-B-a-b.

The line a-b in the stress diagram will be the compression in the auxiliary member a-b, and A-b will be the tension in the column A-b. It should be noted that I' is equal to the algebraic sum of the vertical components of the stresses in a-b and A-b. Next lay off x-a = 3,300 lbs. and complete the polygon a-x-c-b by drawing lines through x and b parallel to the auxiliary truss members x-c and b-c respectively. In like manner determine the stresses at the foot of the knee brace by con-

structing the polygon A-b-c-1; and at the top of the column by constructing the polygon c-x-x-2-1, etc., until the diagram is checked up at C with C-$A = R^1$. The indeterminate case at the joint U_3, can be solved



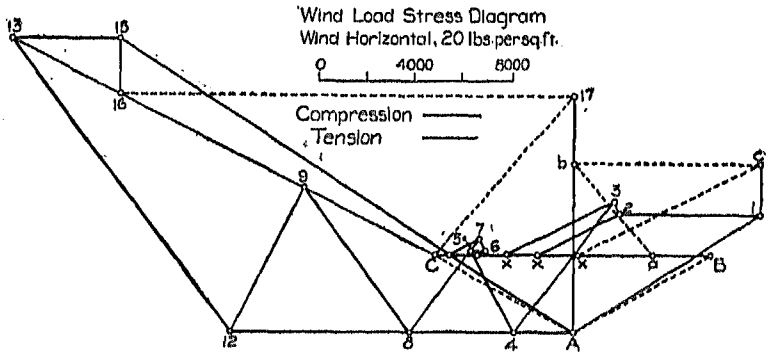


FIG. 54. WIND LOAD STRESS DIAGRAM, CASE 2.

by computing the stress in 5-6 (component due to stress in 6-7), and substituting it in the diagram, or by substituting an auxiliary member.

The stresses in the auxiliary members are represented by dotted lines and are of no value in designing the bent. It should be noted that the auxiliary members do not affect the stresses in the trusses and knee braces, which are correctly given in the stress diagram.

The maximum stress in the knee brace A-15 is compression, and occurs on the leeward side.

The maximum shear in the leeward column below the knee brace is $H^1 = 5600$ lbs.; the maximum shear above the knee brace is 13,100 lbs. The maximum moment occurs at the foot of the knee brace and is $H^1 \times 14 \times 12 = 940{,}800$ inch-lbs.

Case 3. Wind Load Stresses: Wind Horizontal; Columns Fixed at Base.—This is Case 2 with the base of the column hinged at the point of contra-flexure. In calculating H and V, Fig. 55, the wind

See Form 41.

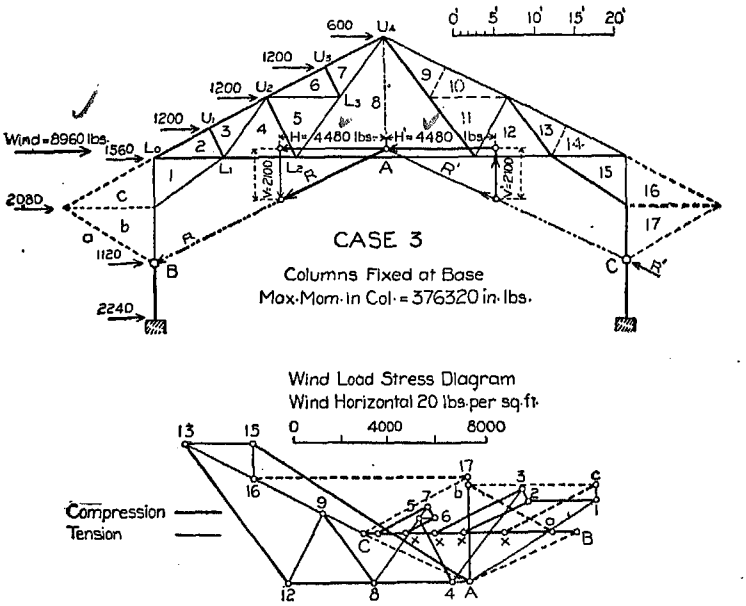
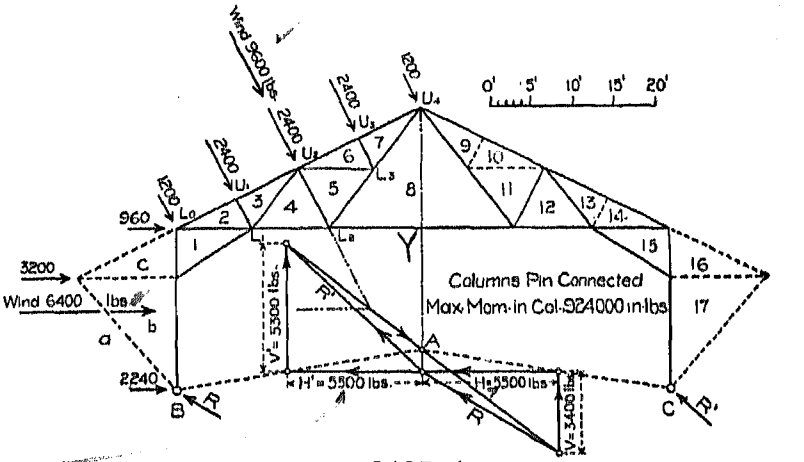


FIG. 55. WIND LOAD STRESS DIAGRAM, CASE 3.

above the point of contra-flexure only (see formula (45)) produces stresses in the bent. The value of fixing the columns at the base is seen by comparing the stresses in Case 2 with those in Case 3, both being drawn to the same scale. Maximum shear in the leeward column below the knee brace is $H^1 = 4480$ lbs.; above the knee brace is 5230 lbs. The maximum positive moment occurs at the foot of the knee brace and negative moment at the foot of the column, and is $H^1 \times 7 \times 12 = 376{,}320$ inch-lbs.

Case 4. Wind Load Stresses: Wind Normal; Columns Hinged. —In Fig. 56 the resultant of the external wind forces on the sides and the roof acts through their intersection, and is parallel to $C\ B$ in the stress diagram (line $C\ B$ is not drawn). To calculate V and V^1 connect the point of intersection, A, of the resultant wind and the vertical line through the center of truss, with the bases of the columns B and C.



CASE 4

Wind Load Stress Diagram

Wind Normal, Roof 18 lbs. Sides 20 lbs. sq. ft.

0 4000 8000

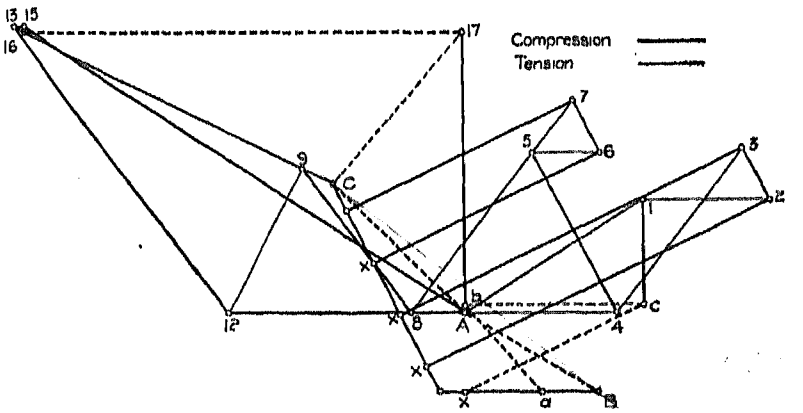
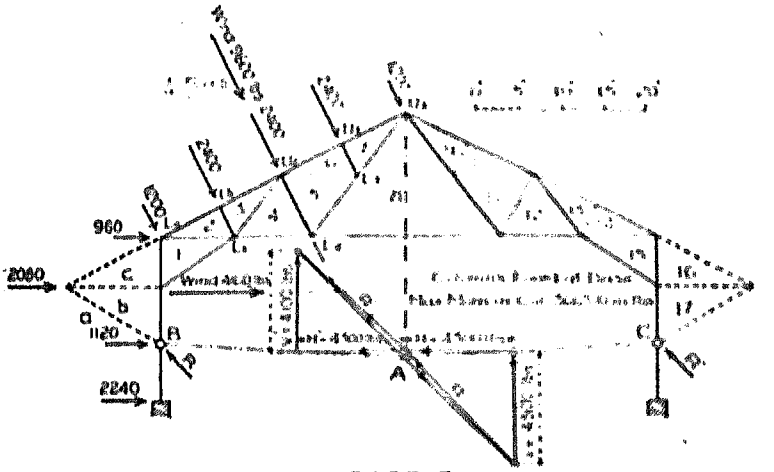


FIG. 56. WIND LOAD STRESS DIAGRAM, CASE 4.

From A lay off one-half of resultant wind on each side, and from the extreme ends drop vertical lines V and V^1 to the dotted lines $A\ B$ and $A\ C$. The vertical lines V and V^1 will be the vertical reactions, the horizontal lines will be H and H^1, and R and R^1 will be the resultants of the horizontal and vertical reactions at B and C respectively. The stresses are calculated by beginning at the base of the column B as in Case 2. In the polygon a-B-A-b at B, A-B = R, a-B = 2240 lbs., and a-b and A-b are the stresses in a-b and A-b respectively.



CASE 5

Wind Load Stress Diagram

Wind Normal Load [illegible] lbs. per sq. ft.

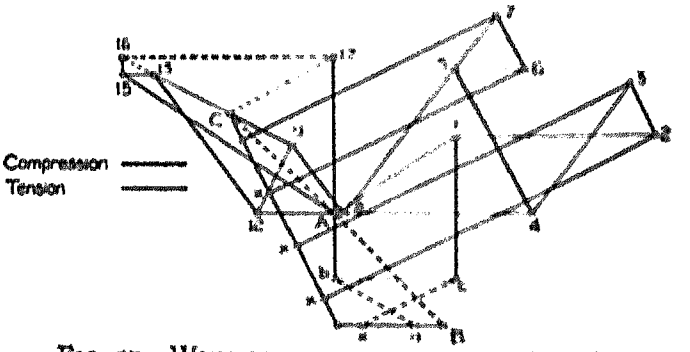


FIG. 57. WIND LOAD STRESS DIAGRAM, CASE 5.

The maximum shear in the leeward column below the knee brace is $H^1 = 5500$ lbs., above the knee brace is 12,800 lbs.; the maximum moment occurs at the foot of the knee brace and is $H^1 \times 14 \times 12 =$ 924,000 inch-lbs.

Case 5. Wind Load Stresses: Wind Normal; Columns Fixed at Base.—This is Case 4 with the base of the column moved up to the point of contra-flexure The maximum shear in the leeward column below the knee brace is 4300 lbs., above the knee brace is 5000 lbs.; the maximum positive moment occurs at the foot of the knee brace and negative moment at the foot of the column and is $H^1 \times 7 \times 12 = 361{,}200$ inch-lbs. For analysis see Fig. 57.

Maximum Stresses.—The stresses in the different members of the bent for the different cases are given in Table V. The maximum

TABLE V.

Name of Piece	Stresses in a Bent For					
	Dead Load	Snow Load	Wind Load			
			Case 2	Case 3	Case 4	Case 5
X-2	+9300	+18900	+3700	+2900	+15400	+14900
X-3	+8800	+17600	+4900	+4000	+15400	+14900
X-6	+8200	+16400	+ 400	+1400	+10200	+11200
X-7	+7700	+15400	+1400	+3400	+10200	+11200
X-9	+7700	+15400	-6100	-1900	- 1400	+2800
X-13	+9300	+18600	-19800	-8600	-14600	- 3600
1-2	-8300	-16600	+ 5700	+2800	- 5100	- 8000
2-3	+1100	+ 2200	+ 500	+ 500	+2400	+2400
3-4	-1200	- 2400	-6800	-4900	- 8500	-6700
4-5	+2200	+4400	+3800	+3000	+7300	+6600
5-6	-1200	- 2400	- 600	- 600	- 2600	- 2600
6-7	+1100	+ 2200	+ 500	+ 500	+2400	+2400
5-8	-2400	-4800	-4300	-3300	- 8200	-7400
7-8	-3600	- 7200	-4900	-3900	-10800	-10000
8-9	-3600	- 7200	+7500	+3600	+ 7400	+3500
9-12	+2200	+4400	-6700	-3200	- 6700	- 3200
12-13	-1200	- 2400	+15200	+7400	+14800	+7000
Y-4	-7100	-14200	+2400	+ 800	- 6000	-7700
Y-8	-4700	- 7400	+6800	+4000	+ 2200	- 400
Y-12	-7100	-14200	+14200	+7700	+ 9700	+2100
13-15	-8300	-16600	+4600	+3000	+ 500	-1300
A-1			-9000	-6200	-8500	-6700
A-15			+22300	+11000	+21500	+10400
A-b	+4800	+9600	- 3200	- 2100	+ 3400	+4500
C-1	+4800	+9600	+1700	+1300	+8000	+7600
A-17	+4800	+9600	+3200	+2100	+5300	+4100
15-16	+4800	+9600	- 8600	- 3800	- 6400	- 2400

stresses in the different members of the bent for (1) dead load plus maximum snow load; (2) dead load plus wind load, Case 4; (3) dead load plus minimum snow load plus wind load, Case 4; and (4) a vertical dead load of 40 lbs. per sq. ft. horizontal projection of the roof are given in Table VI. The stresses which control the design of the members may be seen in Table VI. By comparing these values with the stresses given in the last column the accuracy of the equivalent load method can be seen.

TABLE VI.

Maximum Stresses in a Bent For				
Name of Piece	Dead Load + Max. Snow Load	Dead Load + Wind Load Case 4	Dead Load + Min. Snow Load + Wind Load Case 4	Vert. Dead Load of 40 lbs. per Sq. Ft of Hor. Proj.
X-2	+28200	+24700	+34300	+41200
X-3	+26400	+24200	+33000	+39200
X-6	+26400	+18400	+26600	+32800
X-7	+23100	+11900	+23600	+30800
X-9	+23100	+ 6300	+14000	+30800
X-13	+28200	- 5300	+ 4000	+37200
1-2	-24900	-13400	-21700	-33200
2-3	+ 3300	+ 3300	+ 4600	+ 4400
3-4	- 3600	- 9700	-10900	- 4800
4-5	+ 6600	+ 9500	+11700	+ 8800
5-6	- 3600	- 3800	- 5000	- 4800
6-7	+ 3300	+ 3300	+ 4600	+ 4400
5-8	- 7200	-10600	-13000	- 9600
7-8	-10800	-14400	-18000	-14400
8-9	-10800	+ 3800	+ 200	-14400
9-12	+ 6600	- 4500	- 2300	+ 8800
12-13	- 3600	+13600	+12400	- 4800
Y-4	-21300	-13100	-20200	-28400
Y-8	-14100	- 2500	- 5200	-14800
Y-12	-23100	+ 2600	- 4500	-28400
13-15	-24900	- 7800	-16100	-32400
A-1		- 8500	- 8500	
A-15		+21300	-21300	
A-b	+14400	+ 8200	+13000	+19200
C-1	+14400	+12800	+17600	+19200
A-17	+14400	+10100	+14900	+19200
15-16	+14400	- 1800	+ 3200	+19200

Graphic Calculation of Reactions.—The graphic method for calculating the reactions given in Fig. 56 and Fig. 57 may be proved as follows: In Fig. 57a the intersection of ΣW and the center line of the bent is at A. Draw A–B and A–C, lay off ΣW so that it is bisected by the point A, and draw 1–2 and 3–4. Then 1–2 equals V' and 3–4 equals V as shown in the following proof:

Proof.—To calculate V take moments of external forces about C, and

$$V = \frac{\Sigma W \times b}{L}$$

$$= \frac{4 \text{ times area triangle } A\text{–}4\text{–}C}{L}$$

But area triangle A–4–C is also equal to $\frac{3\text{–}4 \times L}{4}$, and

$$V = \frac{3\text{–}4 \times L}{L} = 3\text{–}4$$

which proves the construction. It may be proved in like manner that $1\text{–}2 = V'$.

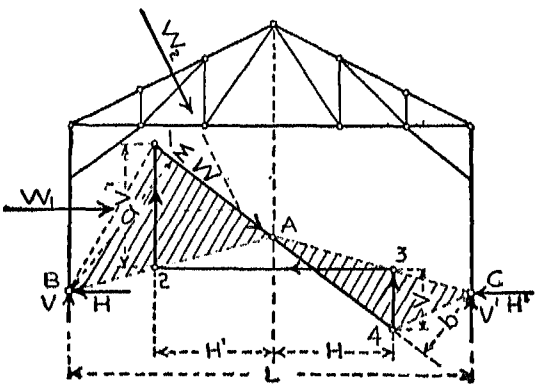


FIG. 57a.

TRANSVERSE BENT WITH VENTILATOR.—The calculation of the wind stresses in a transverse bent with a monitor ventilator is shown in Fig. 57b. The bents are spaced 32′ 0″ centers and

are designed for a horizontal wind load of 20 lbs. per sq. ft., the n mal wind roof load being obtained by Hutton's formula as shown Fig. 6.

The point of contra-flexure is found by substituting in formu (41) to be at a point $y_0 = 17.0'$. The external forces are calculated the bent above the point of contra-flexure by multiplying the area s ported at the point by the intensity of the wind pressure. For exam the load at B is $32' \times 6.75' \times 20$ lbs. = 4320 lbs.

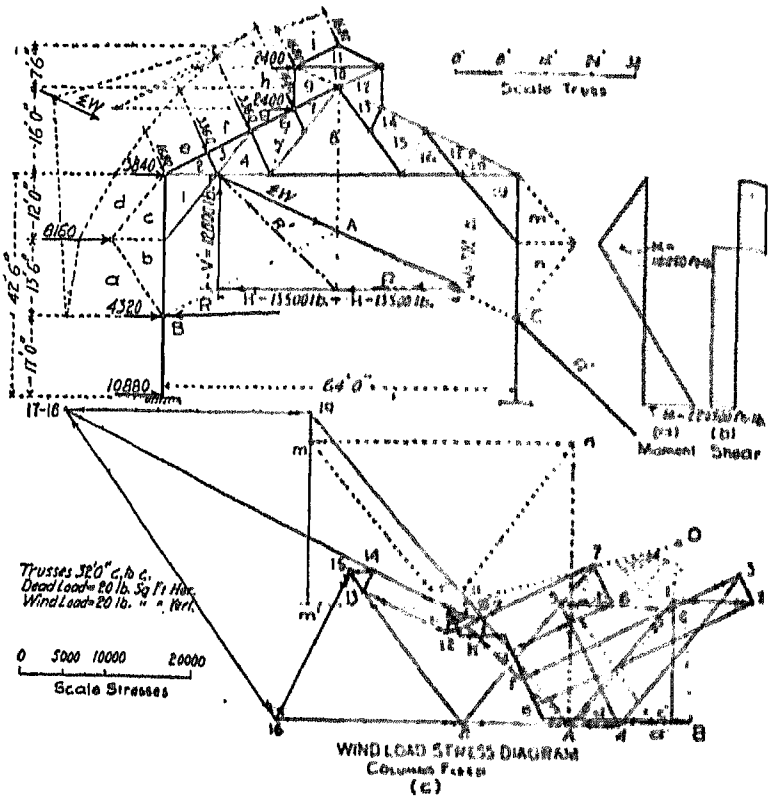


Fig. 57b.

The line of application and the amount of the external wind lo ΣW, is found by means of a force and an equilibrium polygon. Σ acts through the intersection of the strings parallel to the rays O

and O–C, and is equal to C–B (line C–B is not drawn in force polygon) in amount. The reactions R and R' are calculated by the graphic method as previously described.

The calculation of stresses is begun at point B in the windward column, and in the stress diagram the stresses at B are found by drawing the force polygon a–B–1–b–a. The remaining stresses are calculated as for a simple truss. In calculating the stresses in the ventilator it was assumed that diagonals 9–10 and 10–12 are tension members, so that 9–10 will not be in action when the wind is acting as shown. Before solving the stresses at the joint 6–7–9 it was necessary to calculate the stresses in members i–11, 10–11, and 9–h. The remainder of the solution offers no difficulty to one familiar with the principles of graphic statics.

The stress in post b–a is equal to V', while the stress in 1–c is found by extending 1–c to c' in the stress diagram, c' being a point on the load line. The stress in post n–d is equal to V'', while the stress in 19–m is found by extending 19–m to m' in the stress diagram, m' being a point on the horizontal line drawn through C. The kind of stress in the different members is shown by the weight of lines in the bent diagram and by arrows in the stress diagram, one arrow indicating the direction and kind of stress the first time a stress is used and two arrows indicating the second time a stress is used.

TRANSVERSE BENT WITH SIDE SHEDS.—Transverse bents with side sheds are quite often used in the design of shops and mills. The calculation of the stresses due to wind load in a bent of this type is an interesting application of the author's graphic solution of stresses in transverse bents.

It is required to calculate the stresses due to a horizontal wind load of 30 lbs. per square foot on the sides and the normal component of 30 lbs. (Hutton's Formula, Fig. 6) on the roof, the bents being spaced 20′ 0″ centers, as in Fig. 57c. The loads are calculated, and by means of a force polygon in (d) and an equilibrium polygon in (a) the resultant wind ΣW is found to pass through point E, and to be equal to 30,800 lbs.

Calculation of Reactions.—The horizontal shear of 25,400 lbs. will be taken by the columns in proportion to their rigidities, in this

case the rigidities of the columns are assumed equal and the shear at the foot of each column will be 6350 lbs. The vertical reactions will be due to two forces: (1) a vertical load of 17,200 lbs., which will be taken equally by the four columns, making a load of 4300 lbs. on each; and (2) to a bending moment of 25,400 lbs. $\times$ 9.2 ft. $=$ 233,680 ft.-lbs. (the bending moment about C. G. is also equal to 30,800 lbs $\times$ 7.6 ft.), which will be resisted by the columns and will cause reactions varying as the distance from the center of gravity of the columns, (E), as in the case of the continuous portal, Fig. 63.

Let v'_1, v'_2, v'_3, and v'_4 represent the reactions due to moment in the columns, respectively; then if a is the reaction on a column at a units distance from the center of gravity we will have $v'_1 = -a\ 40$, $v'_2 = -a\ 20$, $v'_3 = +a\ 20$, and $v'_4 = +a\ 40$. The resisting moment of each column will be equal to the reaction multiplied by the distance from the center of gravity, and $a\ 40^2 + a\ 20^2 + a\ 20^2 + a\ 40^2 = 233{,}680$ ft.-lbs. from which $a = 58.42$ lbs. and $v'_1 = -2340$ lbs.; $v'_2 = -1170$ lbs.; $v'_3 = +1170$ lbs.; $v'_4 = +2340$ lbs.

Now adding the reactions due to (1) and (2) we have

$$V_1 = 4300 - 2340 = +1960 \text{ lbs.},$$
$$V_2 = 4300 - 1170 = +3130 \text{ lbs.},$$
$$V_3 = 4300 + 1170 = +5470 \text{ lbs.},$$
$$V_4 = 4300 + 2340 = +6640 \text{ lbs.}$$

Combining the horizontal and vertical reactions we have $R_1 = a_1\text{–}A =$ 6600 lbs.; $R_2 = A\text{–}B = 7200$ lbs.; $R_3 = B\text{–}C = 8400$ lbs.; $R_4 = C\text{–}D = 9100$ lbs. These reactions close the force polygon in (d).

Calculation of Stresses.—Auxiliary members are substituted as shown by the broken lines. It will be seen that these members are arranged so that all bending is removed from the columns and that the stresses in the truss members are correctly given in the stress diagram. The calculation is started at point A at the foot of the left-hand column as in the case of the simple transverse bent, and reactions R_2 and R_3 are substituted as the calculation progresses, the stress diagram finally closing at the base of the leeward column, point D. The stresses are given on the members in (a). The direct stresses in the columns are easily found by algebraic resolution beginning at the

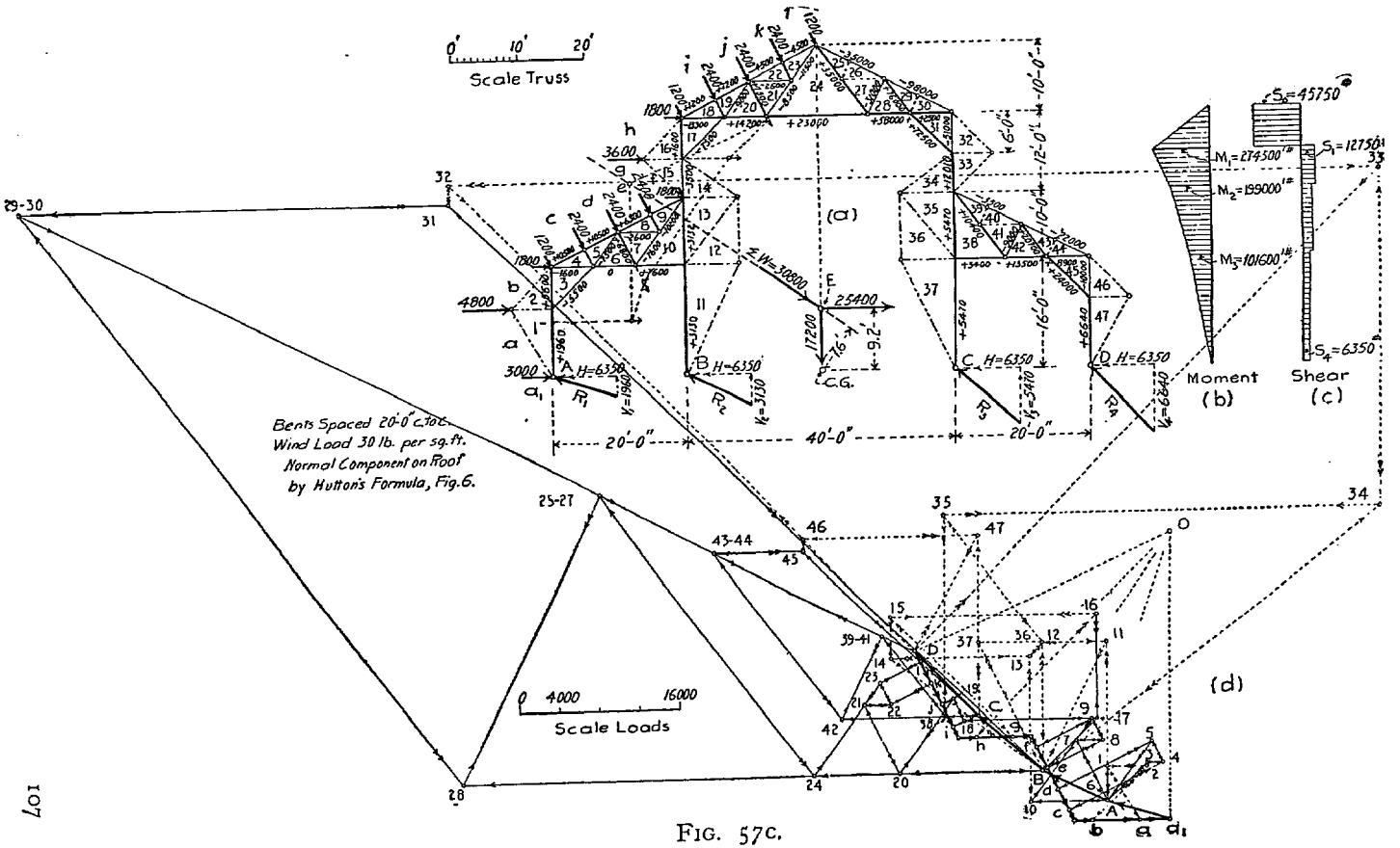

Scale Truss
0' 10' 20'
Bents Spaced 20-0" c. to c.
Wind Load 30 lb. per sq. ft.
Normal Component on Roof
by Hutton's Formula, Fig. 6.
Scale Loads
0 4000 16000
(a)
(b)
(c)
(d)
Moment
Shear
20'-0"
40'-0"
20'-0"
C.G.
A H=6350
B H=6350
C H=6350
D H=6350
E 25400
ΣW=30800
M1=274500"lb
M2=139000"lb
M3=101600"lb
S1=12750
S4=6350
S5=45750

FIG. 57c.

foot of the columns where the direct stress is equal to V_1, V_2, V_3 and V_4, respectively. The stresses in the leeward side of the main truss are very large due to the small depth of truss in line of the member 29–30. The stresses could be materially reduced and a considerable saving of material obtained by using a main truss of type (b) or (h), Fig. 88.

Moment and Shear in Columns.—The bending moment in the main leeward column is shown in (b), the maximum moment is at the foot of the knee brace and is $M_1 = 274{,}500$ ft.-lbs. The shear diagram is shown in (c), the maximum shear is between the foot of the knee brace and the top of the column and is $S_0 = 45{,}750$ lbs.

NOTE.—The stresses in a bent with side sheds obtained by the preceding method are approximate for the reason that the assumed conditions are probably never entirely realized. In the exact calculation of the stresses, of which the above solution is the first approximation, the deformation of the framework is considered in a manner similar to that of the two-hinged arch in Chapter XIV. The approximate solution is entirely adequate for all practical purposes.

CHAPTER XII.

Stresses in Portals.

Introduction.—Portal bracing is frequently used for bracing the sides of mill buildings and open sheds. There are many forms of portal bracing in use, a few of the most common of which are shown in Fig. 58.

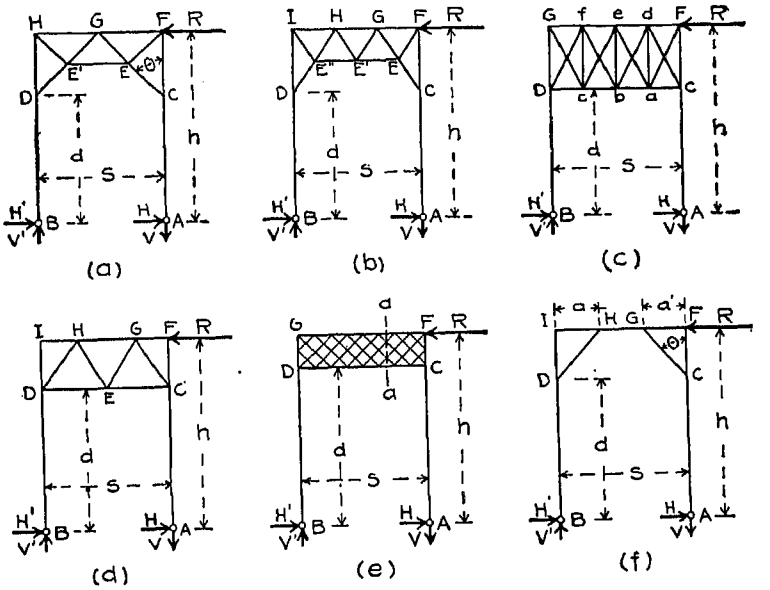


Fig. 58.

Portal bracing may be in separate panels or may be continuous. The columns may be hinged or fixed at the base in either case.

CASE I. STRESSES IN SIMPLE PORTALS: Columns Hinged.—The deflections of the columns in the portals shown in Fig. 58 are assumed to be equal and

$$H = H' = \frac{R}{2}$$

Taking moments about the foot of the windward column

$$V' = -V = \frac{R\,h}{s}$$

Having found the external forces, the stresses in the members may be found by either algebraic or graphic methods.

Algebraic Solution.—*Portal (a).*—To obtain the stress in member $G\,C$, (a) Fig. 58, pass a section cutting $G\,F$, $E\,F$ and $G\,C$, and take moments of the external forces to the right of the section about point F as a center.

$$G\,C = -\frac{H\,h}{(h-d)\sin\theta} \tag{46}$$

But $H = \frac{R}{2}$, and $(h-d)\sin\theta = \frac{s}{2}\cos\theta$. Substituting these values in (46) we have

$$G\,C = -\frac{R\,h}{s\cos\theta} = -V\sec\theta \tag{47}$$

Resolving at C and F we have, stress in $E\,F = 0$, and also stresses $E\,E'$ and $H\,E' = 0$.

To obtain stress in $G\,D$, pass section cutting $H\,G$, $H\,E'$ and $G\,D$, and take moments of the external forces to the left of the section about point H as a center.

$$G\,D = \frac{H\,h}{(h-d)\sin\theta} = +V\sec\theta \tag{48}$$

To obtain stress in $G\,F$, pass a section cutting $G\,F$, $E\,F$ and $G\,C$, and take moments of the external forces to the right of the section about point C as a center.

$$G\,F = +\frac{R\,(h-d)+H\,a}{h-d} \tag{49}$$

To obtain stress in $H\,G$, pass a section cutting $H\,G$, $H\,E'$ and $G\,D$, and take moments of the external forces to the left of the section about the point D as a center.

$$H\,G = -\frac{H\,d}{h-d} \tag{50}$$

The stress in the windward post, $A\,F$, is zero above and V below the foot of the knee brace C; the stress in the leeward post is zero above and V^1 below the foot of the knee brace D.

The shear in the posts is H below the foot of the knee brace, and above the foot of the knee brace is given by the formula

$$S = \frac{H\,d}{h-d} = \text{stress in } H\,G \tag{51}$$

The maximum moment in the posts occurs at the foot of the knee braces C and D and is

$$M = H\,d \tag{52}$$

For the actual stresses, moments and shears in a portal of this type, see Fig. 59.

Portal (*b*).—The stresses in portal (b) Fig. 58, are found in the same manner as in portal (a). The graphic solution of a similar portal with one more panel is given in Fig. 60, which see. It should be noted that all members are stressed in portals (b) and (d).

Portal (*c*).—The stresses in portal (c) Fig. 58, may be obtained (1) by separating the portal into two separate portals with simple bracing, the stresses found by calculating the separate simple portals with a load $= \frac{1}{2}\,R$ being combined algebraically, to give the stresses in the portal; or (2) by assuming that the stresses are all taken by the system of bracing in which the diagonal ties are in tension. The latter method is the one usually employed and is the simpler.

Maximum moment, shear, and stresses in the columns are given by the same formulas as in (a) Fig. 58.

Portal (*e*).—In portal (e) Fig. 58, the flanges $G\,F$ and $D\,C$ are assumed to take all the bending moment, and the lattice web bracing

is assumed to take all the shear. The maximum compression in the upper flange $G\ F$ occurs at F, and is

$$G\ F = + \frac{R\ (h - d) + H\ d}{h - d} \tag{53}$$

The maximum tension in the upper flange $G\ F$ is

$$G\ F = - \frac{H\ d}{h - d} \tag{54}$$

The maximum stress in the lower flange $D\ C$ is

$$D\ C = + \frac{H\ h}{h - d} \tag{55}$$

maximum tension occurring at C, and maximum compression occurring at D.

The maximum shear in the portal strut is V, which is assumed as taken equally by the lattice members cut by a section, as $a\ a$.

Maximum moment, shear, and stresses in the columns are given by the same formulas as in (a) Fig. 58.

Portal (*f*).—The maximum moment in the portal strut $I\ F$ in (f) Fig. 58, occurs at H and G, and is

$$M = + H\ h - V\ a \tag{56}$$

The maximum direct stress in $H\ G$ is $+ H$, and in $I\ H$ is

$$I\ H = - \frac{H\ d}{h - d} \tag{57}$$

The maximum stress in $G\ F$ is given by formula (49).

The maximum shear in girder $I\ F$ is equal to V. The stress in $G\ C$ is $-H\ h \div (h - d) \sin \theta$ = stress $H\ D$.

Portal strut $I\ F$ is designed as a girder to take the maximum moment, shear and direct stress.

Maximum moment, shear, and stresses in the columns are given by the same formulas as in (a) Fig. 58.

Graphic Solution.—To make the solution of the stresses statically determinate, replace the columns in the portal with trussed framework

as in Fig. 59. The stresses in the interior members are not affected by the change and will be correctly given by graphic resolution.

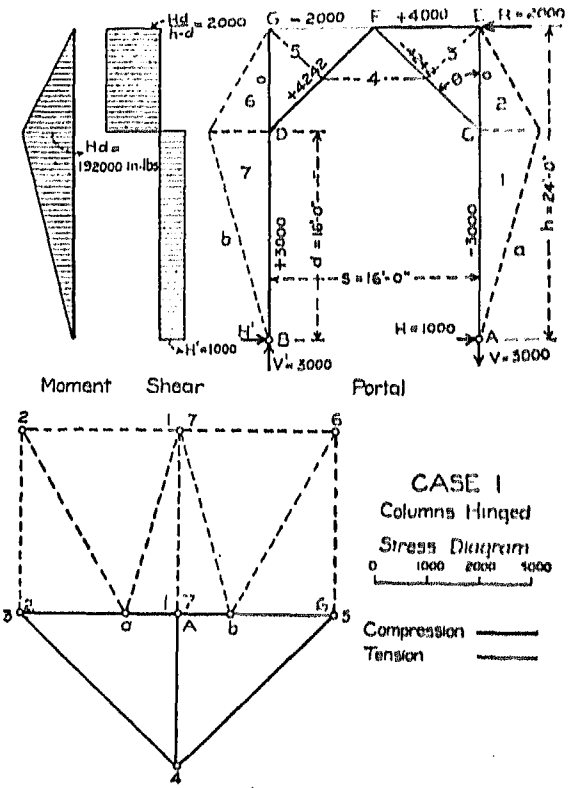


FIG. 59.

As before $H = H^1 = \frac{R}{2}$ and $V = -V^1 = \frac{R\,h}{s}$.

Having the calculated H, H^1, V, and V^1, the stresses are calculated by graphic resolution as follows: Beginning at the base of the column A, lay off A-4 $= V =$ 3000 lbs. acting downward, and A-a $= H =$ 1000 lbs. acting to the right. Then a-1 and 4-1 are the stresses in members a-1 and 4-1, respectively, heavy lines indicating compression and light lines tension. At joint in auxiliary truss to right of C the stress in 1-a is known and stresses in 1-2 and 2-a are found by closing the polygon.

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The stresses in the remaining members are found in like manner, taking joints *C*, *E*, *F*, etc. in order, and finally checking up at the base of the column *B*. The full lines in the stress diagram represent stresses in the portal; the dotted lines represent stresses in the auxiliary members or stresses in members due to auxiliary members, and are of no consequence. The shears and moments are shown in the diagram.

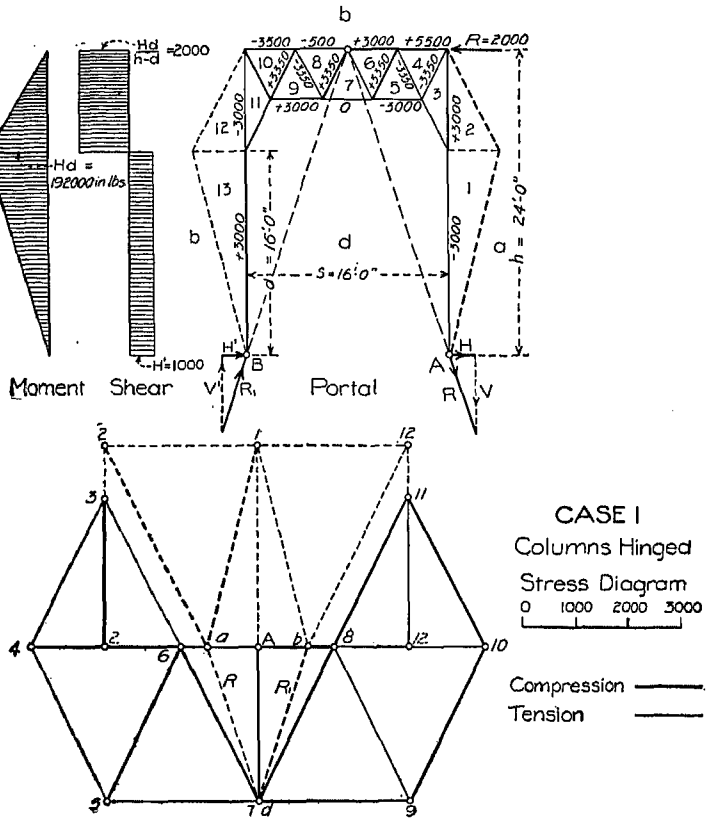


FIG. 60.

Simple Portal as a Three-Hinged Arch.—In a simple portal the resultant reactions and the external load *R* meet in a point at the middle of the top strut, and the portal then becomes a three-hinged arch

(see Chapter XIII), provided there is a joint at that point (point b, Fig. 60).

In Fig. 60 the reactions were calculated graphically and the stresses in the portal were calculated by graphic resolution. Full lines in the stress diagram represent required stresses in the members. Stresses 3-2 and 11-12 were determined by dropping verticals from points 3 and 11 to the load line 4-10.

CASE II. STRESSES IN SIMPLE PORTALS: Columns Fixed.—The calculation of the stresses in a portal with columns fixed at the base is similar to the calculation of stresses in a transverse bent with columns fixed at the base. The point of contra-flexure is at the point

$$y_0 = \frac{d}{2}\left(\frac{d+2h}{2d+h}\right) \tag{41}$$

measured up from the base of the column. The point of contra-flexure is usually taken at a point a distance $\frac{d}{2}$ above the bases of the columns.

The stresses in a portal with columns fixed may be calculated by considering the columns hinged at the point of contra-flexure and solving as in Case I.

Algebraic Solution.—In Fig. 61 we have

$$H = H^1 = \frac{R}{2}$$

and $$V = -V^1 = \frac{R\left(h - \frac{d}{2}\right)}{s}$$

Having found the reactions H and H^1, V and V^1, the stresses in the members are found by taking moments as in (a) Fig. 58, considering the columns as hinged at the point of contra-flexure.

The shear diagram for the columns is as shown in (a) and the moment diagram as in (c) Fig. 61.

Anchorage of Columns.—In order that the columns be fixed, the anchorage of each column must be capable of developing a resisting moment greater than the overturning moment $M = -\frac{Hd}{2}$, shown in

(c) Fig. 61. The anchorage required on the windward side is a maximum and may be calculated as follows: Let T be the tension in the windward anchor bolt, $2a$ be the distance center to center of anchor

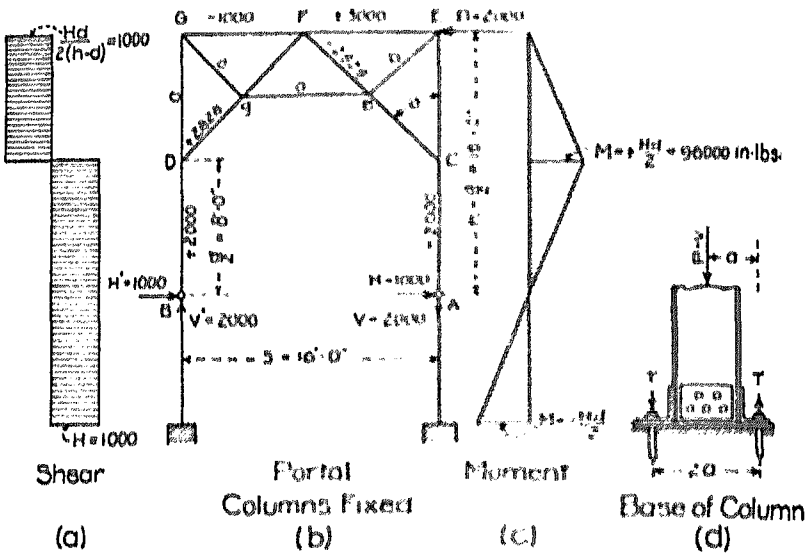


FIG. 61.

bolts, and P be the direct load on the column. Taking moments about the leeward anchor bolt we have

$$2Ta - (P - P')a + \frac{Hd}{2} = 0$$

$$T = -\frac{Hd}{4a} + \frac{P - P'}{2} \qquad (58)$$

If the nuts on the anchor bolts are not screwed down tight, there will be a tendency for the column to rotate about the leeward edge of the base plate, and both anchor bolts will resist overturning.

The maximum pressure on the masonry will occur under the leeward edge of the base plate and will be

$$S = \frac{W'}{A} + \frac{Me}{I}$$

where W = direct stress in post;

A = area of base of column in sq. ins.;

M = bending moment = ½ Hd;

c = one-half the length of the base plate;

I = moment of inertia of the base plate about an axis at right angles to the direction of the wind.

Graphic Solution.—The stresses in the portal in Fig. 62 have been calculated by graphic resolution. This problem is solved in the same manner as the simple portal with hinged columns in Fig. 59.

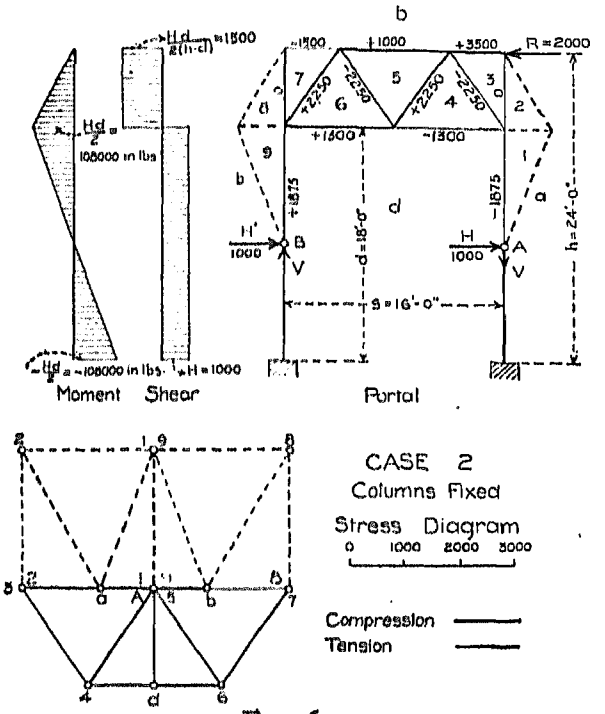


FIG. 62.

STRESSES IN CONTINUOUS PORTALS.—The portal with five bays shown in Fig. 63 will be considered. The columns will all be assumed alike and the deformation of the framework will be neglected. The shears in the columns at the base will be equal, and will

be $$H = \frac{K}{6}.$$

To find the vertical reactions proceed as follows: Determine the center of gravity of the columns by taking moments about the base of one of the columns. Now there will be tension in each one of the columns on the windward side and compression in each one of the columns on the leeward side of the center of gravity of the columns. The sum of the moments of the reactions must be equal to the moment

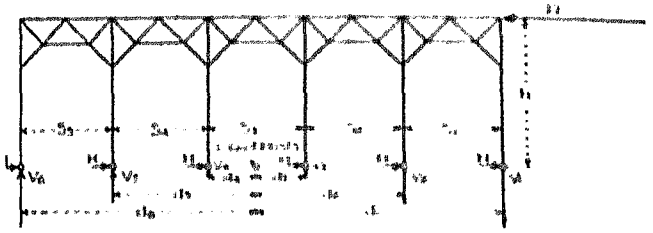


FIG. 63.

of the external wind load, K. The reactions at the bases of the columns will vary as the distance from the center of gravity and their moments will vary as the square of the distance from the center of gravity. Now, if a equals the reaction of a column at a units distance from the center of gravity, we will have $V_1 = -a\,d_1$, $V_2 = -a\,d_2$, $V_3 = -a\,d_3$, $V_4 = +a\,d_4$, $V_5 = +a\,d_5$, and $V_6 = +a\,d_6$

and the moment

$$M = a\,(d_1^2 + d_2^2 + d_3^2 + d_4^2 + d_5^2 + d_6^2) = R\,h$$

$$a\,\Sigma\,d^2 = R\,h$$

$$a = \frac{R\,h}{\Sigma d^2} \qquad (59)$$

Having found a, the vertical reactions may be found.

Now having found the external forces H and V, the stresses can be calculated by either algebraic or graphic methods.

Stresses in a Double Portal.—To illustrate the general problem the stresses in a double portal are calculated by graphic resolution in Fig. 64. In this case

$$H = H = H = \frac{R}{3} = 1000 \text{ lbs.}$$

and $$V = -V^1 = \frac{R\,h}{2\,s} = 2250 \text{ lbs.}$$

The vertical reaction of the middle column is zero. By substituting the dotted members as shown, the stresses can be calculated as in the case of the simple portal. The full lines represent stresses in the portal members. The shear in the columns is equal and is H below, and $\frac{H\,d}{h-d}$ above the foot of the knee brace.

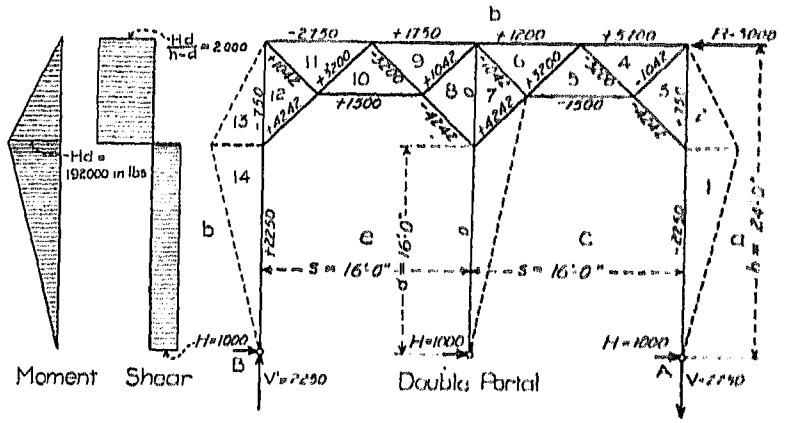


FIG. 64.

The maximum bending moment occurs at the foot of the knee brace and is

$$M = H\,d = 192{,}000 \text{ inch-lbs.}$$

CHAPTER XIII.

Stresses in Three-hinged Arch.

Introduction.—An arch is a structure in which the reactions are inclined for vertical loads. Arches are divided, according to the number of hinges, into three-hinged arches, two-hinged arches, one-hinged arches, and arches without hinges or continuous arches. Three-hinged arches are in common use for exposition buildings, train sheds and other similar structures. Two-hinged arches are rarely used in this country; continuous arches are used only in dome construction.

A three-hinged arch is made up of two simple beams or trusses. Trussed three-hinged arches, only, will be considered in this chapter, and trussed two-hinged arches in the next.

CALCULATION OF STRESSES.—The reactions for a three-hinged arch can be calculated by means of simple statics with slightly more work than that necessary to obtain the reactions in simple trusses. Having determined the reactions the stresses may be calculated by the ordinary algebraic and graphic methods used in the solution of the stresses in simple roof trusses.

Calculation of Reactions: Algebraic Method.—Let H and V, H^1 and V^1 be the horizontal and vertical reactions at the left and right supports for a concentrated load P, placed at a distance x from the center hinge C in the three-hinged arch in Fig. 65.

From the three fundamental equations of equilibrium

$$\Sigma \text{horizontal components of forces} = 0 \qquad (a)$$

$$\Sigma \text{vertical components of forces} = 0 \qquad (b)$$

$$\Sigma \text{moments of forces about any point} = 0 \qquad (c)$$

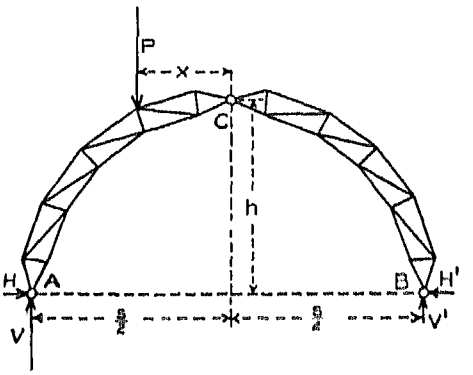


FIG. 65.

we have $$H = H^1$$

and $$V + V^1 = P$$

Taking moments about B, we have

$$Vs - P\left(\frac{s}{2} + x\right) = 0 \tag{60}$$

and taking moments about center hinge C, we have

$$V\frac{s}{2} - Hh - Px = 0 \tag{61}$$

Solving (60) we have

$$V = \frac{P}{s}\left(\frac{s}{2} + x\right) \tag{62}$$

and $$V^1 = P - V = \frac{P}{s}\left(\frac{s}{2} - x\right) \tag{63}$$

Substituting (61) in (62), we have

$$H = H^1 = \frac{P}{2h}\left(\frac{s}{2} - x\right) \tag{64}$$

The horizontal reactions at the crown are the same as at the supports. Reactions for an inclined load may be found by substituting the proper moment arms.

Calculation of Reactions: Graphic Method.—Let P, Fig. 66, be the resultant of all the loads on the left segment. Since there is no

bending moment at hinge C, the line of action of the reaction R_2 must pass through the hinge at the crown. This determines the direction of reaction R_2, and since the three external forces R_1, R_2 and P produce equilibrium in the structure they must meet in a point. Therefore to find the direction of R_1 produce $B\,C$ to d and join d and A.

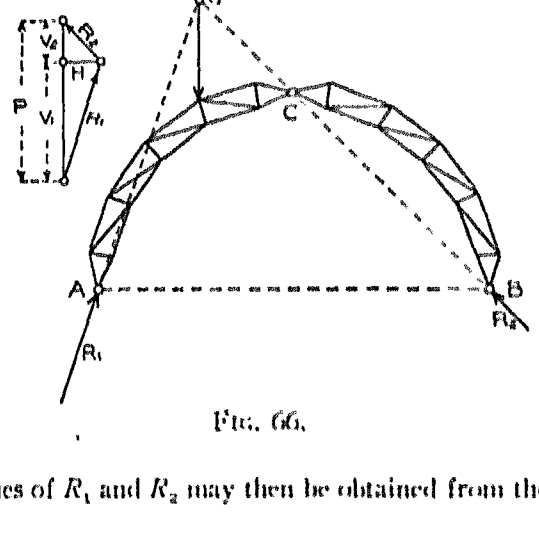


FIG. 66.

The values of R_1 and R_2 may then be obtained from the force polygon.

The reactions due to loads on the right segment may be found in the same manner. The two operations may be combined in one as illustrated in the solution of the dead load stresses in a three-hinged arch, Fig. 67.

Calculation of Dead Load Stresses.—To find the reactions for the dead loads in Fig. 67, the loads are laid off on the load line of the force polygon in order, beginning at the left reaction A, and two equilibrium polygons, one for each segment, are drawn using the same force polygon. The vertical reactions at the crown, P_c, and at abutments, P_a and P_b, are found by drawing a line through pole o of the force polygon parallel to the closing lines of the equilibrium polygons. The load P_c at the crown causes reactions R_1^1 and R_2^1, and combining

reactions R_1^1 and P_n at A, and R_2^1 and P_B at B, we have the true reactions R_1 and R_2.

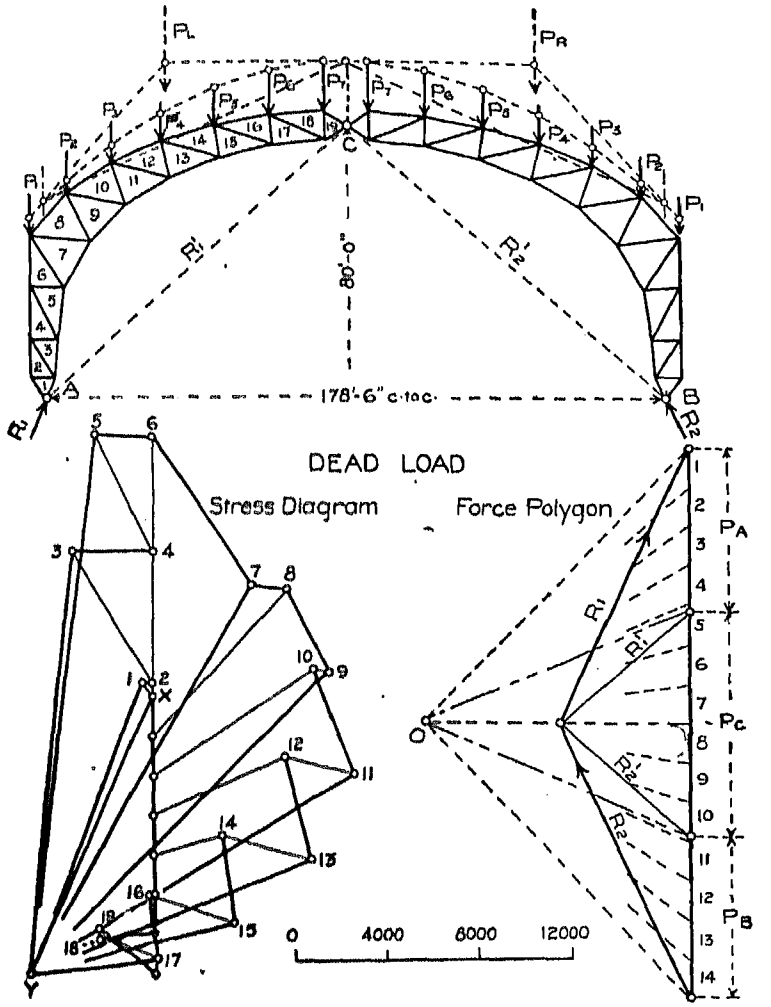


FIG. 67.

Having obtained the reactions, the stresses in the members are found in the same manner as in simple trusses. In Fig. 67 the stresses in the left segment are calculated by graphic resolution. The diagram

is begun with the left reaction x-$y = R_1$. Where the dead load is symmetrical a stress diagram need only be drawn for one segment.

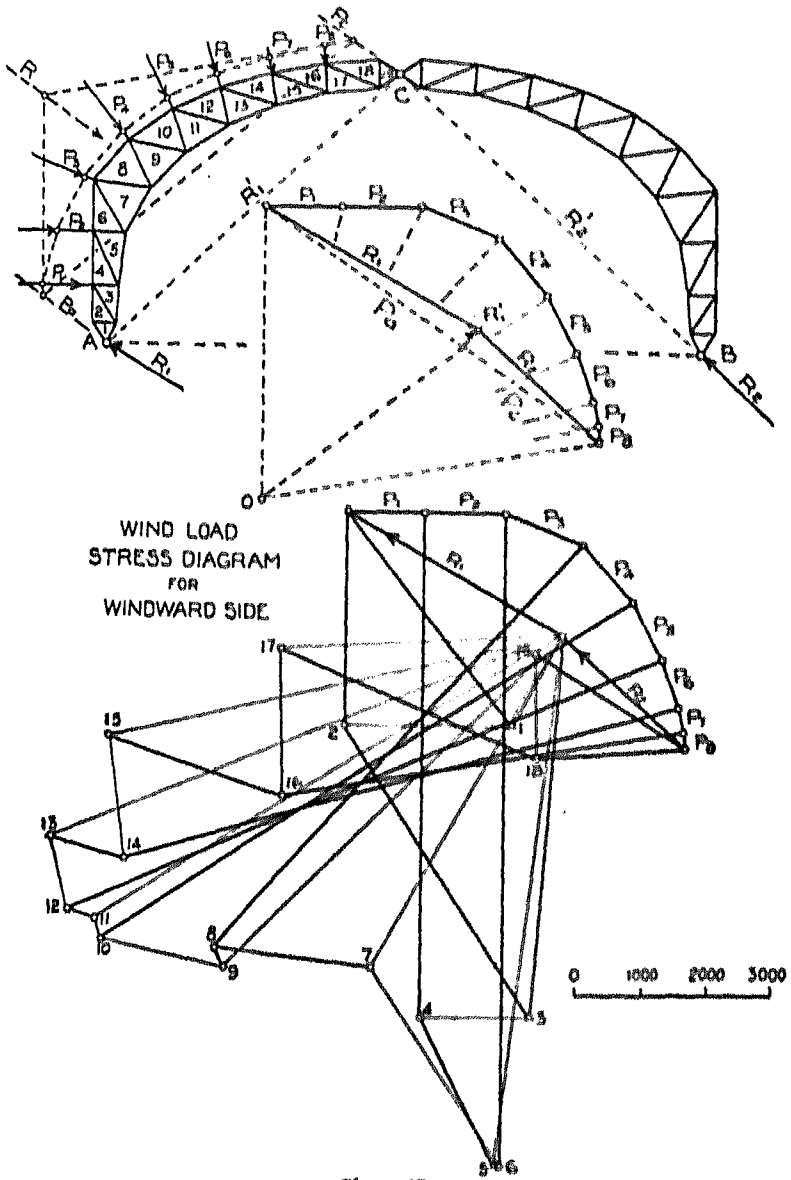


FIG. 68.

Calculation of Wind Load Stresses.—The reactions for wind load in Fig. 68 are found as follows:

The reactions P_a and P_c for the windward segment, considering it a simple truss supported at the hinges, are found by means of force

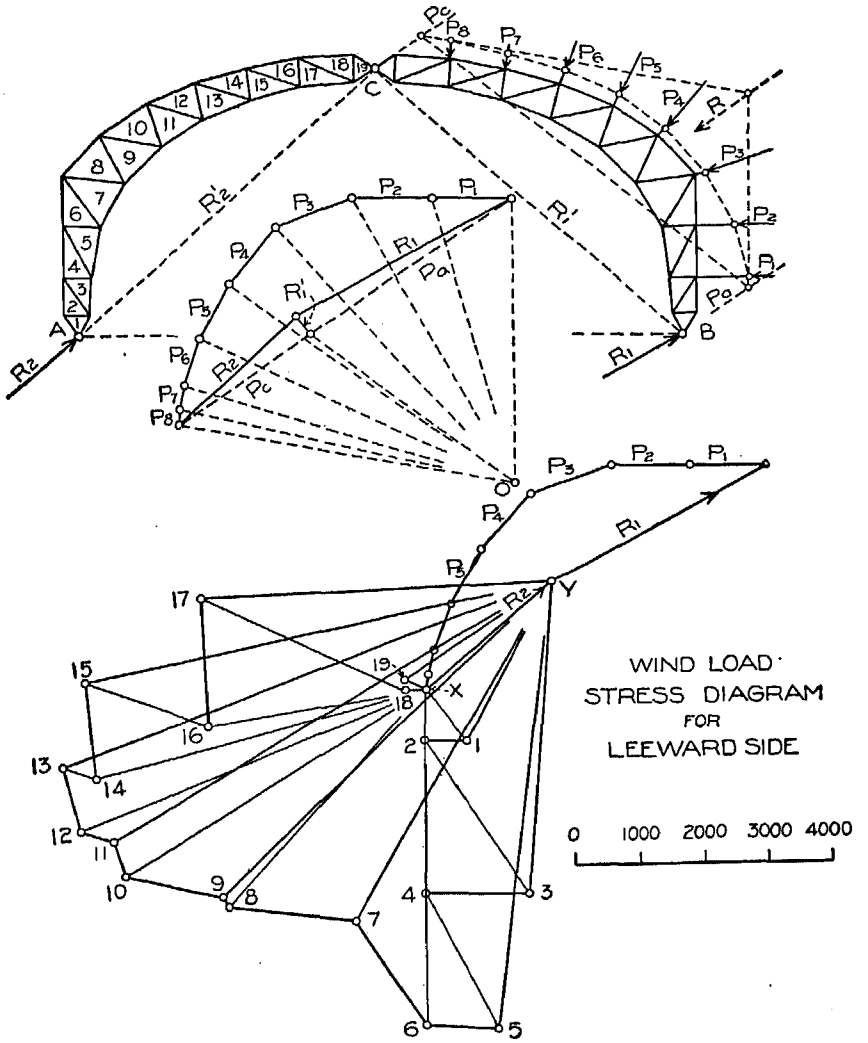


FIG. 69.

and equilibrium polygons. The lines of action of P_a and P_c are parallel to each other and to the resultant R. The line of action of the right reaction, R_2, must pass through the center hinge C, and the reaction P_0 will be replaced by two reactions R_2 and R_1^1 parallel to R_2^1 and R_1^1 in the arch respectively, and the force triangle will be closed by drawing R_1 in the force polygon. The intersection of force R and reactions R_1 and R_2 falls outside the limit of the diagram.

Having obtained the reactions, the stresses in the members are calculated in the same manner as in a simple truss.

The wind load stresses must be calculated in both the windward and leeward segments. The wind load stress diagram for the windward segment is shown in Fig. 68, and for the leeward segment in Fig. 69, compression being indicated in the stress diagrams by heavy lines and tension by light lines. Both wind load stress diagrams and the dead load stress diagram are usually constructed for the same segment of the arch. By comparing wind load stress diagrams in Fig. 68 and Fig. 69, it will be seen that there are many reversals in stress. The maximum stresses found by combining the dead, snow and wind load stresses as in the case of simple trusses and transverse bents, are used in designing the members.

CHAPTER XIV.

Stresses in Two-hinged Arch.

Introduction.—A two-hinged arch is a frame-work or beam with hinged ends which has inclined reactions for vertical loads. The bottom chords of two-hinged arches are usually cambered, however, a simple truss becomes a two-hinged arch if the ends are fixed to the abutments so that deformation in the direction of the length of the truss is prevented.

The horizontal components of the reactions may be supplied either by the abutments or by a tie connecting the hinges. In the latter case the deformation of the tie must be considered in determining the horizontal reactions. Two-hinged arches are statically indeterminate structures and their design is subject to the same uncertainties as continuous and swing bridges.

Two-hinged roof arches are rigid and economical, but have been used to a very limited extent on account of the difficulties experienced in their design. The methods outlined in this chapter are quite simple in principle, although they necessarily require quite extended calculations. Two-hinged roof arches with open framework, only, will be considered in this chapter.

CALCULATION OF STRESSES.—The vertical reactions in a two-hinged arch are the same as in a simple truss or a three-hinged arch having the same loads and span. The horizontal reactions, however, depend upon the deformation of the framework and cannot be determined by simple statics alone. Before the deformations can be calculated, the sizes of the members must be known, and conversely, before the sizes

of the members can be calculated, the stresses which depend upon the deformations must be known. Any method for the calculation of the stresses in a two-hinged arch is, therefore, necessarily a method of successive approximations. With a skilled computer, however, it is rarely necessary to make more than two or three trials before obtaining satisfactory results in designing roof arches. Two-hinged bridge arches require somewhat more work to design than roof arches on account of the greater number of conditions for maximum stresses in the members.

Having determined the correct value of the horizontal thrust, H, the stresses in a two-hinged arch may be calculated by the ordinary algebraic or graphic methods used in the solution of the stresses in simple trusses.

Calculation of the Reactions.—In Fig. 70 the vertical reactions, V_1 and V_2, are the same as for a simple truss. The horizontal reactions, H, will be equal and will be the forces which would prevent change in length of span if the ends of the arch were free to move. The horizontal thrust, H, will therefore be the force which, applied at the roller end of a simple truss, will prevent deformation and make the truss a two-hinged arch.

An expression for H may be determined as follows: In Fig. 70 assume that all members are rigid except the member 1-y, which is increased in length δ, under the action of the external load, W. The

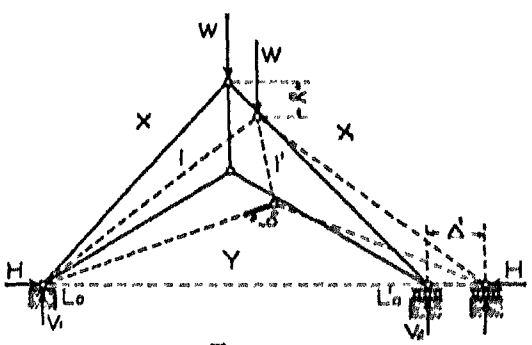


FIG. 70.

movement of the truss Δ' at the hinge L'_0 will then be due to the change in length, δ, of the member 1-y.

Let h^1 be the horizontal reaction necessary to bring L^1_0 back to its original position, and let $U\ h^1$ be the stress in the member 1-y due to the horizontal thrust h^1. Now the internal work, $\frac{1}{2}\ \delta\ h^1\ U$, in shortening the member 1-y to its original length will be equal to the external work, $\frac{1}{2}\ h^1\ \Delta'$, required to bring the hinge $L_0{}^1$ back to its original position,

$$\frac{1}{2}\ h^1\ \Delta' = \frac{1}{2}\ \delta\ h^1\ U$$

and

$$\Delta' = \delta\ U \tag{65}$$

but $\delta = \frac{P\,L}{E}$, where P is the unit stress in the member 1-y due to the external load W, L is the length of the member 1-y, and E is the modulus of elasticity of the material of which the member is composed. Substituting this value of δ in (65) we have

$$\Delta' = \frac{P\,U\,L}{E} \tag{66}$$

where U is the stress in 1-y due to a load $\frac{h^1}{h^1} =$ unity at L^1_0

Now if each one of the remaining members of the arch is assumed to be distorted in turn, the others meanwhile remaining rigid, the distortion in each case at L^1_0 will be represented by the general equation (66) and the total deformation, Δ, at L^1_0 will be

$$\Delta = \Sigma\frac{P\,U\,L}{E} \tag{67}$$

Let $P^1\ h^1$ be the unit stress in the member 1-y due to a horizontal thrust h^1, then by the same reasoning

$$\Delta' = \delta\ U \tag{65a}$$

but

$$\delta = \frac{P^1\ h^1\ L}{E}$$

and

$$\Delta' = \frac{h^1\ P^1\ U\,L}{E}$$

and the total deformation, Δ, will be

$$\Delta = \Sigma \frac{h^1 P^1 U L}{E} = H \Sigma \frac{P^1 U L}{E} \tag{68}$$

Now for equilibrium, the values of Δ as given in equations (67) and (68) must be equal, and we have, after solving for H

$$H = \frac{\Sigma \frac{P U L}{E}}{\Sigma \frac{P^1 U L}{E}} \tag{69}$$

which is an expression for computing the horizontal thrust in any two-hinged arch due to external loads. This formula holds for any system of loading as long as P is the unit stress due to that loading, U is the stress in the member and P^1 is the unit stress in the member due to a unit load acting at the point at which the deformation is desired, and parallel to the direction in which the deformation is to be measured.

The method of finding the correct value of the horizontal reaction, H, is as follows: (1) calculate the stresses in the arch for the given loading on the assumption that it is a simple truss with one end supported on frictionless rollers; (2) calculate the stresses in the arch for an assumed horizontal reaction, $H^1 =$, say, 20000 lbs. on the assumption that it is a simple truss on frictionless rollers; (3) calculate the deformation, Δ, of the free end of the truss for the given loads by means of formula (67); (4) calculate the deformation, Δ' of the free end of the truss for the assumed horizontal reaction $H^1 = 20000$ lbs. by means of formula (68). The true value of H is then by formula (69) given by the proportion

$$H : H^1 :: \Delta : \Delta'$$

$$H = \frac{H^1 \Delta}{\Delta'} = \frac{20000\ \Delta}{\Delta'} \tag{70}$$

The calculation of the horizontal reaction, H, and the stresses can be made by algebraic methods alone or by a combination of graphic and algebraic methods. The first requires less work, while the second

is probably easier to understand. The algebraic solution will be given first.

Algebraic Calculation of Reactions.—In Table VII the values of the unit stress, P, in each member due to the external loads are given in column 5; values of $\frac{PL}{E}$ are given in column 6; values of the stress, U, in each member due to a unit horizontal thrust are given in column 8; and values of $\frac{PUL}{E}$ are given in column 9. The algebraic sum of the quantities in column 9 gives the total deformation, $\Delta = .956$ inches at the point where the unit horizontal thrust was applied measured parallel to the line of action of the thrust.

TABLE VII.

Simple Truss with Vertical Loads

1 Member	2 Area Sq. in.	3 Length, L Inches	4 Stress lbs.	5 Unit Stress P lbs.	6 $\frac{PL}{E}$	7 No. of Mem.	8 U	9 $\frac{PUL}{E}$
1-X	5.3	252	+60000	+11320	+.095	9	-0.90	-.086
2-X	5.3	192	+41000	+ 7740	+.050	6	-0.80	-.040
4-X	5.3	180	+67000	+12650	+.076	1	-1.45	-.110
2'-X	5.3	192	+41000	+ 7740	+.050	12	-0.80	-.040
1'-X	5.3	252	+60000	+11320	+.095	13	-0.90	-.086
1-Y	5.3	216	-25000	- 4720	-.034	8	+1.60	-.054
3-Y	5.3	192	-37000	-10760	-.069	4	+2.05	-.141
3'-Y	5.3	192	-37000	-10760	-.069	10	+2.05	-.141
1'-Y	5.3	216	-25000	- 4720	-.034	14	+1.60	-.054
1-2	2.0	150	-30000	-15000	-.075	7	+0.75	-.056
2-3	4.0	204	+32000	+ 8000	+.054	5	-0.45	-.024
3-4	4.0	150	-22000	- 5500	-.028	2	+0.80	-.022
3'-4	4.0	150	-22000	- 5500	-.028	3	+0.80	-.022
2'-3'	4.0	204	+32000	+ 8000	+.054	11	-0.45	-.024
1'-2'	2.0	150	-30000	-15000	-.075	13	+0.75	-.056
				Total Deformation $= \sum \frac{PUL}{E} =$				.956

In Table VIII similar values are given for the arch as a truss with an assumed horizontal reaction of $H' = 20000$ lbs. The algebraic sum of the quantities in column 9 gives the total deformation, $\Delta' = .574$ inches at the point where the horizontal thrust was applied.

TABLE VIII.

Simple Truss with H = 20000 lbs.

1 Member	2 Area Sq in	3 Length inches	4 Stress lbs	5 Unit Stress f lbs	6 f L / E	7 No of Mem	8 U	9 P U L / E
1-X	5.5	252	-18000	-3400	.0428	2	-.090	.025
2-X	5.5	192	-16000	-3000	.030	6	-.080	.015
4-X	5.5	180	-29000	-5300	.055	1	-.145	.048
2'-X	5.5	192	-16000	-3000	.030	6	-.080	.015
1'-X	5.5	252	-18000	-3400	.0428	2	-.090	.025
1-Y	5.5	216	+37000	+6900	+.043	8	+.160	.069
3-Y	5.5	192	+41000	+7700	+.050	4	+.205	.102
3'-Y	5.5	192	+41000	+7700	+.050	10	+.205	.102
1'-Y	5.5	216	+37000	+6900	+.043	14	+.160	.069
1-2	20	150	+15000	+1500	+.018	1	+.075	.029
2-3	40	204	-9000	-2250	-.015	3	-.045	.007
3-4	40	150	+16000	+4000	+.020	2	+.080	.016
3'-4	40	150	+16000	+4000	+.020	3	+.080	.016
2'-3'	40	204	-9000	-2250	-.015	11	-.045	.007
1'-2'	20	150	+15000	+1500	+.018	13	+.075	.029

Total Deformation = $\Sigma \frac{PUL}{E}$ = .574

TABLE IX.

Simple Truss with Dead and Wind Loads

1 Member	2 Area Sq in	3 Length inches	4 Stress lbs	5 Unit Stress f lbs	6 f L / E	7 No of Mem	8 U	9 P U L / E
1-X	5.5	252	+41000	+7500	+.094	2	-.090	-.124
2-X	5.5	192	+32000	+6000	+.075	6	-.080	-.069
4-X	5.5	180	+55000	+10000	+.125	1	-.145	-.155
2'-X	5.5	192	+32000	+6000	+.075	6	-.080	-.090
1'-X	5.5	252	+42000	+7600	+.094	2	-.090	-.105
1-Y	5.5	216	+59000	+10900	+.138	8	+.160	-.125
3-Y	5.5	192	+87000	+16400	+.205	4	+.205	-.215
3'-Y	5.5	192	+74000	+14000	+.175	10	+.205	-.185
1'-Y	5.5	216	+50000	+9250	+.116	14	+.160	-.064
1-2	20	150	+36000	+18000	+.225	1	+.075	-.067
2-3	40	204	+28000	+7000	+.088	5	-.045	-.022
3-4	40	150	+22000	+5500	+.069	2	+.080	-.022
3'-4	40	150	+42000	+10500	+.131	3	+.080	-.042
2'-3'	40	204	+46000	+11500	+.144	11	-.045	-.035
1'-2'	20	150	+42000	+21000	+.263	13	+.075	-.079

Total Deformation = $\Sigma \frac{PUL}{E}$ = 1357

The correct value of H is given by the proportion

$$H : H^1 :: \Delta : \Delta'$$

$$H = \frac{20000 \times .956}{.574} = 33400 \text{ lbs.}$$

In Table IX the deformation, Δ, for the same arch considered as a simple truss and acted upon by dead and wind loads is 1.357 inches, and

$$H = \frac{20000 \times 1.357}{.574} = 47300 \text{ lbs.}$$

Graphic Calculation of Reactions.—In the graphic solution of the horizontal reactions the total amount of the deformations, Δ and Δ' are found by means of deformation diagrams. Before constructing the deformation diagrams the quantities in the first seven columns in Tables VII and VIII or VIII and IX must be

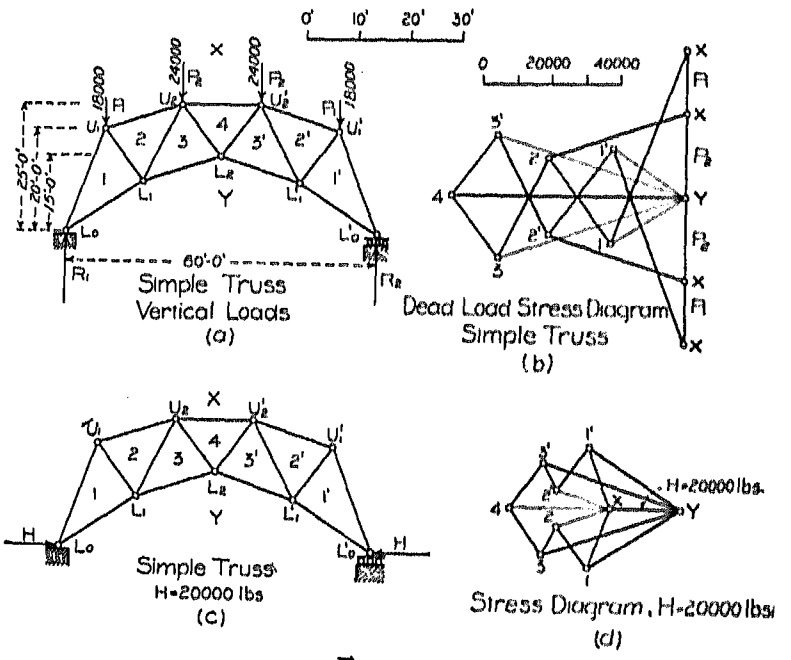


FIG. 71.

calculated. The stresses given in column 4 are calculated as shown in Fig. 71. Column 6, giving deformations of each member, and column 7, giving the order in which these deformations are used, are, however, the only values used in constructing the deformation diagrams.

Deformation Diagram.—The principle upon which the construction of the deformation diagram is based is as follows: Take the two members *a-c* and *c-b* in (*d*) Fig. 72, meeting at the point *c*. Assume that *a-c* is shortened and *b-c* is lengthened the amounts indicated. It is required to find the new position, *c'*, of the point *c*. With center at *a* and a radius equal to the new length of *a-c* = *a-c'*, describe an arc. The new position of *c* must be some place on this arc. Then with a center at *b* and a radius equal to the new length of *b-c* = *b-c'*, describe an arc cutting the first arc in *c'*. The new position of *c* must be some place on this arc and will therefore be at the intersection of the two arcs, *c'*. Since the deformations of the members are always very small as compared with the lengths of the members, the arcs may be replaced by perpendiculars, and the members themselves need not be drawn, (e) Fig. 72.

To draw the deformation diagram, (b) Fig. 72, for the arch as a truss with one end on frictionless rollers and loaded with vertical loads, proceed as follows: Begin with the member marked 1, lay off its deformation = + .076 inches (Table VII., column 6) to scale and parallel to member 1. Now lay off the deformation of 2 = — .028 inches away, from the joint U_2 and parallel to the member 2, and lay off deformation of 3 = — .028 inches, away from the joint U'_2 and parallel to the member 3. Perpendiculars erected at the ends of deformations 2 and 3 will meet in the new position of L_2. The vertical distance between the deformation 1 and point L_2 represents to scale the change in position of L_2 relative to the member $U_2\ U^1_2$. At L_2 in the deformation diagram lay off deformation of 4 = — .069 inches, away from the joint and parallel to the member 4, and at U_2 lay off deformation of 5 = + .054 inches, toward the joint and parallel to the member 5. The perpendiculars erected at the ends of the deformations 4 and 5

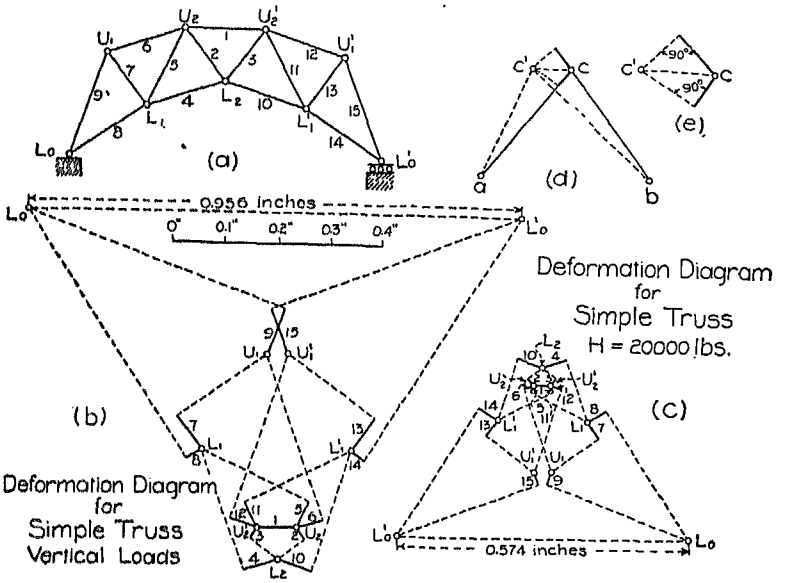


FIG. 72.

determine the new position of joint L_1 relative to the other points. In like manner perpendiculars erected at the ends of deformations 6 and 7 determine U_1, and finally perpendiculars erected at the ends of deformations 8 and 9 determine L_0. The deformation diagram for the right half of the truss is constructed in the same manner. The horizontal line joining L_0 and L^1_0 represents to scale the movement of the joint L^1_0.

In drawing the deformation diagram it is immaterial whether plus deformations are laid off toward the joints and minus deformations away from the joints as was done in the preceding problem, or the reverse. The former method is more common, but the latter is probably more consistent. The deformation diagram (*b*) if drawn in the latter way would be turned upside down and inside out.

Calculation of Dead Load Stresses in Arch.—In Fig. 71, (*b*) is the stress diagram for the arch as a simple truss with vertical loads as shown in (*a*); and (*d*) is the stress diagram for the arch as a simple

truss with a horizontal thrust, H^1, of 20000 lbs. as shown in (*c*). The quantities for calculating the deformations of the simple truss with vertical loads are given in Table VII, and the deformation diagram is shown in (*b*) Fig. 72. The quantities for calculating the deformations of the simple truss with a horizontal thrust of 20000 lbs. are given in Table VIII, and the deformation diagram is shown in (*c*) Fig. 72. The true value of H is found by the proportion

$$H : 20000 : : .956 : .574$$

$$H = 33400 \text{ lbs.}$$

The stress diagram for the two-hinged arch with $V = V' = 42000$ lbs., and $H = H = 33400$ lbs. is shown in (*b*) Fig. 73.

The difference in the stresses in the members of a simple truss and a two-hinged arch may be seen by comparing stress diagram (*b*) Fig. 71, and stress diagram (*b*) Fig. 73, both diagrams being drawn

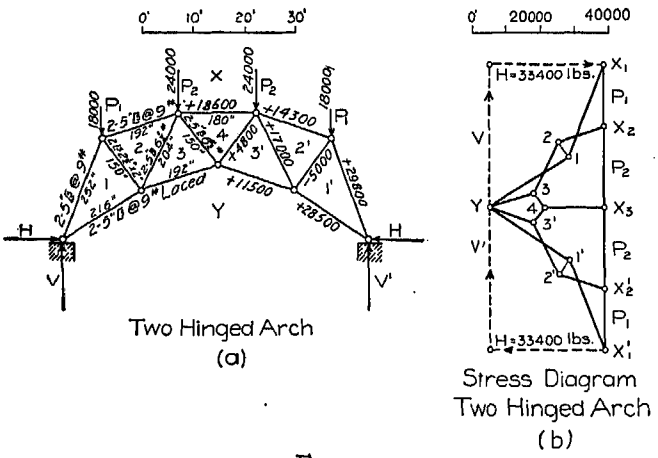


FIG. 73.

to the same scale. The stresses in the arch may be found from the stresses given in Tables VII and VIII by adding the stresses in column 4, Table VII, to the corresponding stresses in column 4, Table VIII, multiplied by 1.67, the ratio between the actual and as-

sumed horizontal reactions. For example, the stress in 1-x in the arch equals $+ 60000 - 18000 \times 1.67 = + 29800$ lbs. Stress in 1-y equals $- 25000 + 32000 \times 1.67 = + 28440$ lbs.

Dead and Wind Load Stresses in Arch.—In Fig. 74, (b) is the stress diagram for the arch as a simple truss loaded with dead and wind loads as shown in (a). Table IX gives the same data for this case as

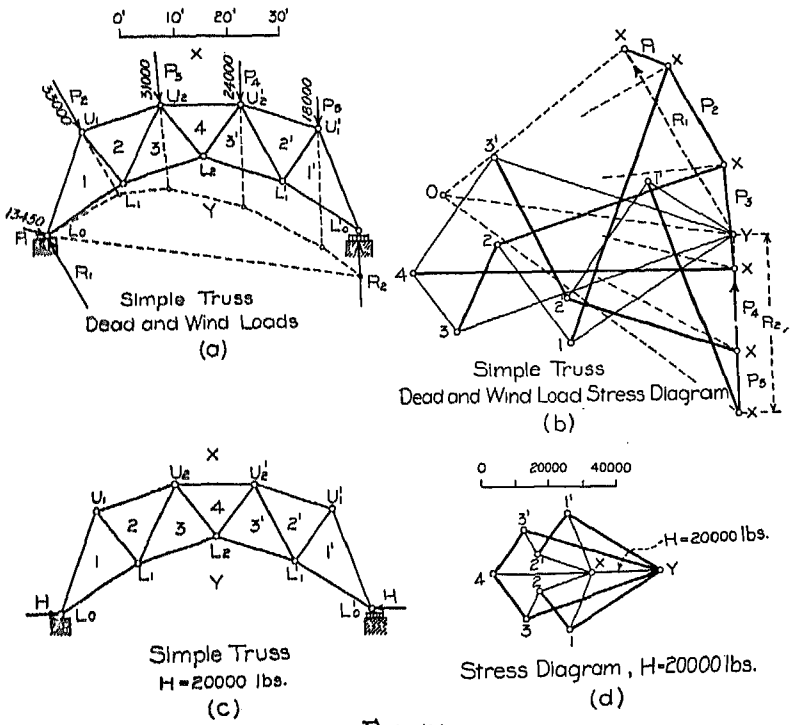


FIG. 74.

are given in Table VII for the simple truss with vertical loads. The deformation diagram for the deformations given in column 6, Table IX, is shown in (b) Fig. 75. In drawing the deformation diagram for this case the member marked 1 was assumed to be fixed in position and the other members were assumed free to move. The horizontal distance between L_0 and L'_0 will be the total deformation required.

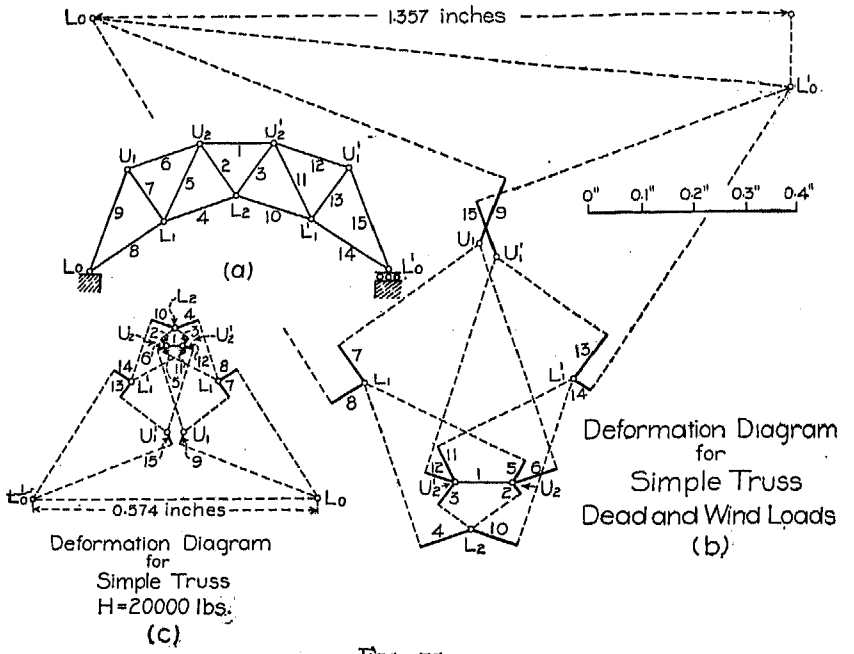


FIG. 75.

The deformation diagram for the simple truss with a horizontal thrust, H^1, of 20000 lbs. is given in (*c*) Fig. 75 and is the same as that given in (*c*) Fig. 72.

The true value of H is found by the proportion

$$H : 20000 :: 1.357 : .574$$

$$H = 47300 \text{ lbs.}$$

The stress diagram for the two-hinged arch with dead and wind loads and a horizontal thrust, H, of 47300 lbs. is given in (*b*) Fig. 76. The stresses in the arch for this case may be found from the stresses in Tables IX and VIII by adding the stresses in column 4, Table IX, to the corresponding stresses in column 4, Table VIII, multiplied by 2.865, the ratio between the actual and assumed horizontal reactions.

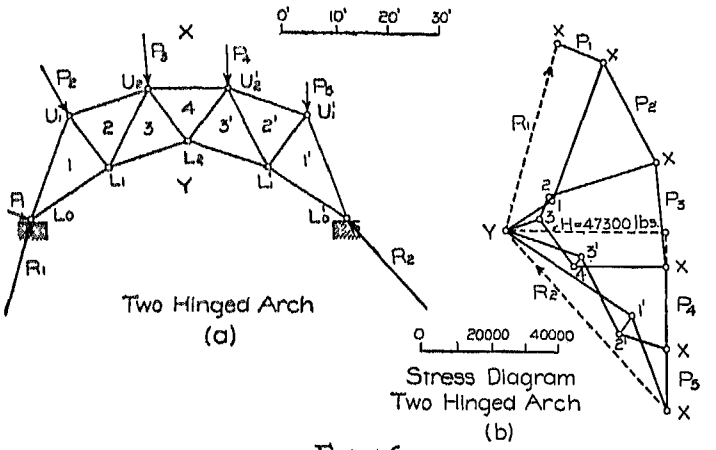


FIG. 76.

As a check on the accuracy of the calculations the movement at L_0' in the arch was calculated in Table X and was found to be zero as it should be.

Arch With Horizontal Tie.—If a horizontal tie is used the final deformation of the arch will be equal to the deformation of the tie.

TABLE X.

Two Hinged Arch with Dead and Wind Loads

1 Member	2 Area Sq.in.	3 Length, L inches	4 Stress lbs.	5 Unit Stress P lbs.	6 $\frac{PL}{E}$	8 U	9 $\frac{PUL}{E}$
1-X	5.3	252	+43500	+8220	+.069	-0.90	-.063
2-X	5.3	192	+34200	+6450	+.041	-0.80	-.033
4-X	5.3	180	+26500	+5000	+.030	-1.45	-.045
2'-X	5.3	192	+14200	+2660	+.017	-0.80	-.014
1'-X	5.3	252	+29500	+5550	+.047	-0.90	-.042
1-Y	5.3	216	+18000	+3400	+.024	+1.60	+.038
3-Y	5.3	192	+10000	+1890	+.012	+2.05	+.025
3'-Y	5.3	192	+23000	+4350	+.028	+2.05	+.057
1'-Y	5.3	216	+46000	+8700	+.062	+1.60	+.099
1-2	2.0	150	- 500	- 250	-.001	+0.75	-.001
2-3	4.0	204	+ 6500	+1625	+.011	-0.45	-.005
3-4	4.0	150	+15800	+3950	+.020	+0.80	+.016
3'-4	4.0	150	- 4200	-1050	-.005	+0.80	-.004
2'-3'	4.0	204	+22800	+5700	+.039	-0.45	-.018
1'-2'	2.0	150	- 5000	- 2500	-.013	+0.75	-.010
			Total Deformation		$= \Sigma \frac{PUL}{E}$		$= .000$

Assume that the joints L_0 and L_0' in (a) Fig. 73 are connected by a tie having 3 sq. in. cross-section. A force of 1000 lbs. will stretch the tie

$$\Delta' = \frac{1000 \times 720''}{3 \times 29{,}000{,}000} = .0083 \text{ inches.}$$

The movement for 1000 lbs. applied as H is equal to $\frac{.574}{20} = .0287$ inches. The value of H therefore which will produce equilibrium for the arch with vertical loads will be

$$+ .0083\ H + .0287\ H = .956 \times 1000 \text{ lbs.}$$

$$H = \frac{.956 \times 1000}{.0370} = 25840 \text{ lbs.}$$

The stresses in the arch for this case may be found from the stresses in Table VII and Table VIII as previously described.

Temperature Stresses.—Where a horizontal tie is used and all parts of the structure are exposed to the same conditions and range of temperature, the entire arch will contract and expand freely and temperature stresses will not enter into the calculations. Where the tie is protected and where rigid abutments are used the temperature stresses must receive careful attention.

The deformation Δ' due to a uniform change of temperature of t degrees Fahr. when the arch is assumed to be a truss supported on frictionless rollers, will be etL, where e is the coefficient of expansion of steel per degree Fahr. $= .00000665$; t equals change in temperature in degrees Fahr.; and L equals the length of the span.

For a change of 75 degrees Fahr. from the mean, the deformation will be

$$\Delta' = \pm .00000665 \times 75\ L$$

$$= \pm \frac{L}{2000} \qquad (71)$$

For the arch in Fig. 73

$$\Delta' = \pm \frac{720''}{2000} = \pm .36 \text{ inches}$$

This will be equivalent to a change in H of $\pm \frac{.36}{.574} \times 20000 = \pm$ 12540 lbs. The stresses due to temperature in the two-hinged arch will be equal to the stresses in column 4, Table VIII, multiplied by $\pm$ .627. The maximum stresses due to external loads and temperature will be found by adding algebraically the temperature stresses to the stresses due to the external loads. If the arch is not erected at a mean temperature this fact must be taken into account in setting the pedestals.

Design of Two-Hinged Arch.—In designing a two-hinged roof arch proceed as follows: (1) With one end free to move, calculate the stresses in the arch as a simple truss; (2) with an assumed horizontal reaction, H', of, say, 20000 lbs., calculate the stresses in the arch as a simple truss; (3) calculate the stresses in the arch for some assumed value of H, this value of H may be guessed at or often may be estimated with considerable accuracy; (4) design the members for approximate stresses in the arch; (5) calculate the deformation of the arch as a truss for the approximate sections and stresses; (6) calculate the deformation of the arch as a truss for an assumed horizontal reaction of 20000 lbs.; (7) determine a more accurate value of H from the deformations as previously described; (8) recalculate the stresses in the arch, redesign the members, recalculate the deformations, recalculate a new value of H, etc., until satisfactory sections are obtained. The second approximation is usually sufficient. Corrections for horizontal tie and temperature should be applied in making the approximations. The gross area of the sections of all members should be used in determining the deformation of the members. If riveted tension members are much weakened, a somewhat smaller value of E, say, 26,000,000, may be used than the 29,000,000 commonly used for the compression members.

The method just described is much more expeditious than the usual method of designing the members for the stresses found by con-

sidering the arch a simple truss with allowable stresses, say, twice those to be finally allowed. In the latter case the first approximation is usually worthless on account of the reversal of stresses in the members which have been designed as tension members. If the value of H in the first method is taken large enough to make members in compression that the designer's judgment or experience says should be in compression, the second approximation is usually final.

CHAPTER XV.

Combined and Eccentric Stresses.

Combined Direct and Cross Bending Stresses. — Thus far members of trusses and frameworks have been considered as acted on by direct forces, parallel to the axis of the member. While this is the more common case, it often becomes necessary to design members which support loads as in (a), (b), (c), or (d), Fig. 77, or in which the line of action of the external force does not coincide with the neutral axis of the member, (e), (f), (g), or (h), Fig. 77.

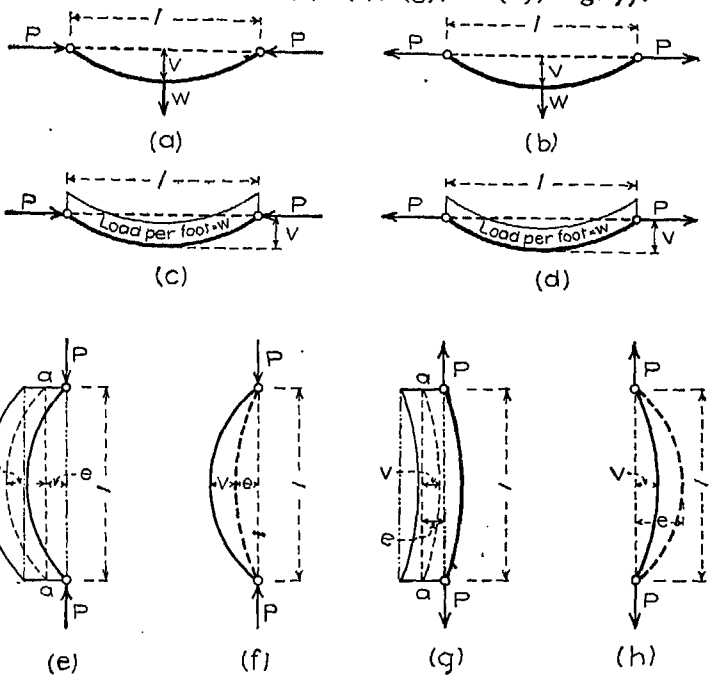


FIG. 77.

The following nomenclature will be used:

Let P = total direct loading on member in pounds;

l = length of member in inches;

L = length of member in feet;

I = moment of inertia of member;

y_1 = distance from neutral axis to remote fibre on side for which stress is desired;

E = modulus of elasticity of the material;

e = eccentricity of P, *i.e.* distance from line of action of P to neutral axis of member in inches;

v = deflection of member in inches;

A = area of member in square inches;

f_1 = fibre stress due to cross bending;

$f_2 = \frac{P}{A}$ = direct fibre stress;

M = total bending moment;

M_2 = bending moment due to deflection, $= P\,v$;

M_1 = bending moment due to external forces and is equal to $\frac{1}{4}\,W\,l$ in (a) and (b); $\frac{1}{8}\,w\,l^2$ in (c) and (d); and $P\,e$ in (e), (f), (g) and (h) Fig. 77.

Now
$$M = M_2 \pm M_1 = Pv \pm M_1 = \frac{f_1 I}{y_1} \qquad (72)$$

But
$$v = \frac{f_1\ l^2}{c\,E\,y_1}$$

in which c is a constant depending upon the condition of the ends, and the manner in which the beam is loaded.

Substituting this value of v in (72) we have

$$\frac{P f_1\ l^2}{c\,E y_1} \pm M_1 = \frac{f_1 I}{y_1}$$

and reducing, the stress due to cross bending is

$$f_1 = \frac{M_1 y_1}{I \pm \frac{P l^2}{c E}} \tag{73}$$

where the minus sign is to be used when P is compression and the plus sign is to be used when P is tension.

The factor c may be taken equal to 10 for columns, beams and bars with hinged ends as in Fig. 77; equal to 24 where one end is hinged and the other end is fixed; and equal to 32 where both ends are fixed.

The total stress in the member due to direct stress and cross bending will therefore be for columns with hinged ends

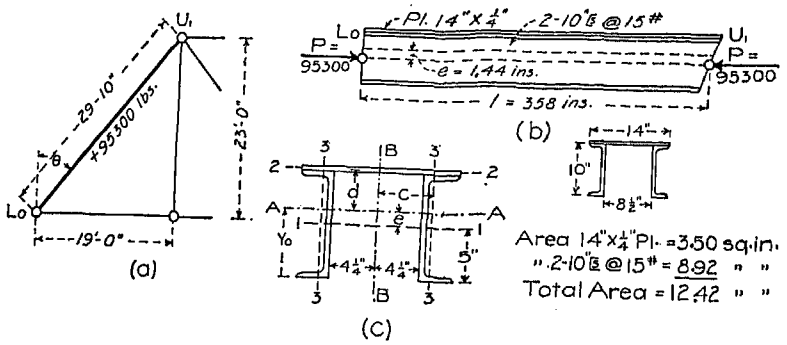
$$f_1 + f_2 = \frac{M_1 y_1}{I \pm \frac{P l^2}{10 E}} + \frac{P}{A} \tag{74}$$

Formula (74) is general, and applies to all forms of sections and all forms of loading. The second term in the denominator is minus when P is compression, and plus when P is tension.

In finding the stress due to weight of member and direct loading, the value for f_1 given by formula (73) must be multiplied by the sine of the angle that the member makes with a vertical line.

Combined Compression and Cross Bending.—The method of calculating direct and cross bending stresses will be illustrated by calculating the stresses in the end post of a bridge due to direct compression, weight, eccentricity of loading, and wind moment.

The end post is composed of two 10-inch channels weighing 15 lbs. per foot with a 14″ × ¼″ plate riveted on the upper side and laced on the lower side with single lacing. The pins are placed in the center of the channels giving an eccentricity of $e = 1.44$ inches. The compressive stress P produces a uniform compression on all fibres of the section; weight of the member causes tension on the lower and compression on the upper fibres; eccentricity of the load P causes compression on lower and tension on upper fibres; and wind moment causes com-



To locate neutral axis A-A take moments about lower edge of channels

$$Y_o = \frac{8.92 \times 5 + 3.5 \times 10.125}{12.42} = 6.44"$$

Eccentricity, e = 6.44 - 5.00 = 1.44"

Moment of Inertia, I_A, about A-A

Let $I_{[}$ = I of [about axis 1-1 = 133.8
$I_{pl.}$ = I of Pl. about axis 2-2 = .02
$A_{[}$ = Area of [= 8.92 sq.in.
$A_{pl.}$ = Area of Pl. = 3.50 sq.in.

Then $I_A = I_{[} + A_{[}e^2 + I_{pl.} + A_{pl.}d^2$
$= 133.8 + 8.92 \times (1.44)^2 + .02 + 3.5(3.685)^2$
$= 199.8$

Radius of gyration, $r_A = \sqrt{\frac{199.8}{12.42}} = 4.0"$

Moment of Inertia, I_B, about B-B

Let $I_{[}$ = I of [about axis 3-3 = 4.6
$I_{pl.}$ = I of Pl. about axis B-B = 57.17
$A_{[}$ = Area of [= 8.92 sq.in.
$A_{pl.}$ = Area of Pl. = 3.50 sq.in.

Then $I_B = I_{[} + A_{[}(4.25 + .64)^2 + I_{pl.}$
$= 4.6 + 8.92(4.89)^2 + 57.17$
$= 275.0$

Radius of gyration, $r_B = \sqrt{\frac{275.0}{12.42}} = 4.7"$

STRESS DUE TO WEIGHT OF MEMBER

The total weight of the member is as follows:—

2-10"[@15# - 30'-0' long	=	900 lbs.
1-14"x¼" Pl. @ 11.9# 30'-0' "	=	357 "
Details and Lacing - 26%	=	328 "
Total weight, W_1	=	1585 "

Bending Moment due to weight of the member, $M = \frac{1}{8} W l \sin\theta$

Stress due to weight, $f_w = \frac{M Y_1}{I - \frac{Pl^2}{10E}} = \frac{\frac{1}{8} W l \sin\theta . Y_1}{I - \frac{Pl^2}{10E}}$

Stress due to weight in upper fibre

$$f_w = \frac{\frac{1}{8} \times 1585 \times 358 \times .633 \times 3.81}{199.8 - \frac{95300 \times 358^2}{10 \times 28\,000\,000}} = +1100 \text{ lbs. (compression)}$$

Stress due to weight in lower fibre

$$f_w' = \frac{6.44}{3.81} \times 1100 = -1860 \text{ lbs. (tension)}$$

STRESS DUE TO ECCENTRIC LOADING

The stress in the extreme fibre due to eccentric loading will be

$$f_e = \frac{My_1}{I - \frac{Pl^2}{10E}} = \frac{Pey_1}{I - \frac{Pl^2}{10E}}$$

Eccentric stress in upper fibre

$$f_e = \frac{95300 \times 1.44 \times 3.81}{199.8 - \frac{95300 \times 358^2}{10 \times 28\,000\,000}} = -3347 \text{ lbs.(tension)}$$

Eccentric stress in lower fibre

$$f_e' = \frac{6.44}{3.81} \times 3347 = +5657 \text{ lbs.(compression)}$$

The resultant stress due to eccentric loading and weight will be

$$f_1 = f_e + f_w$$

$$= -3347 + 1100 = -2247 \text{ lbs. in upper fibre.}$$

$$= +5657 - 1860 = +3797 \text{ lbs. in lower fibre.}$$

The maximum stress in the member due to direct loading, weight of member and eccentric loading will occur in the lower fibre and will be

$$f_2 + f_1 = \frac{P}{A} + f_e + f_w$$

$$= \frac{95300}{12.42} + 3797 = +11470 \text{ lbs}$$

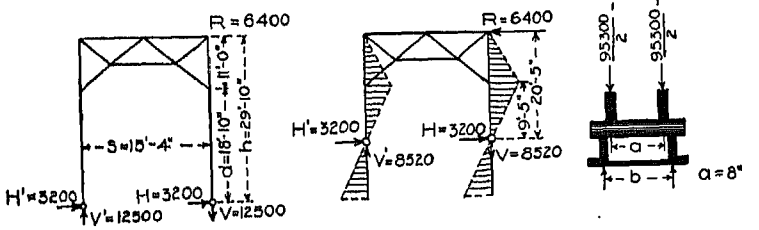
To determine the position of the pin so that stress due to weight will neutralize stress due to eccentric loading make

$$Pe' = \frac{1}{8} Wl \sin\theta$$, where e' is the distance of the pin below the neutral axis.

Substituting and solving, $95300 \times e' = \frac{1}{8}(1585 \times 358 \times .633)$

$$e' = .48''$$

STRESS DUE TO WIND MOMENT



Portal, Columns Hinged. (a)

Portal, Columns Fixed. (b)

Windward Pedestal. (c)

Before calculating the stress due to wind moment, it will be necessary to determine whether the end post is fixed or hinged.

If the end post is fixed, the negative moment developed at the lower pin will be $M = \frac{Hd}{2} = \frac{3200 \times 226}{2} = 361600$ in lbs.

In order to obtain this condition of fixidity, the stress in the member must develop a resisting moment equal in amount.

Therefore the post may be considered fixed if

$$\left(\frac{95300 - V}{2}\right) a \geq \frac{Hd}{2}$$

or $$\left(\frac{95300 - 8520}{2}\right) 8 \geq 361600$$

but $347120 < 361600$ and the end post will not be fixed.

(While this is the usual solution, the resisting moment certainly reduces the bending moment and the bending stress is less than computed below.)

The maximum moment will then occur at the foot of the portal knee brace and will be $M = 3200 \times 226 = 723200$ in lbs.

Stress due to wind moment is a maximum in the leeward post and is

$$f_w = \frac{My_1}{I - \frac{Pl^2}{10E}} = \frac{723200 \times 7}{27.5 - \frac{(95300 + 12500)\,358^2}{10 \times 28000000}}$$

$$f_w = \pm 22480 \text{ lbs.}$$

Stress, f_w, is compression on the windward side and tension on the leeward side of the member.

pression on the windward and tension on the leeward fibres. The maximum fibre stress will come at the foot of the knee brace either on the upper or lower fibres on the windward side of the post, depending upon whether the stress due to weight is greater than the stress due to eccentric loading, or the reverse. In this case the maximum stress comes on the lower fibres of the windward side of the post.

Combined Tension and Cross Bending.—The stress due to cross bending when the member is also subjected to direct tension is given by the formula

$$f_1 = \frac{M_1 y_1}{I + \frac{P l^2}{c E}} \tag{75}$$

the nomenclature being the same as in (74). The constant c is taken equal to 10 where the ends are hinged.

Stress in a Bar Due to its Own Weight:—

Let b = breadth of bar in inches;

h = depth of bar in inches;

w = weight of bar per lineal inch = 0.28 b h';

$f_2 = \frac{P}{b\,h}$ = direct unit stress.

We will also have

$y_1 = \frac{1}{2}\ h$;

$M_1 = \frac{1}{8}\ w\ l^2$;

$P = f_2\ b\ h$.

Substituting in (75) we have

$$f_1 = \frac{\frac{1}{8}\ w\ l^2\ \frac{h}{2}}{\frac{b\,h^3}{12} + \frac{f_2\,b\,h\,l^2}{10 \times 28{,}000{,}000}}$$

$$= \frac{4{,}900{,}000\ h}{f_2 + 23{,}000{,}000\left(\frac{h}{l}\right)^2} \qquad (76)$$

where f_1 is the extreme fibre stress in the bar due to weight, and is tension in lower fibre and compression in upper fibre.

If the bar is inclined, the stress obtained by formula (76) must be multiplied by the sine of the angle that the bar makes with a vertical line.

Formula (76) is much more convenient for actual use than formula (75).

Diagram for Stress in Bars Due to Their Own Weight.—Taking the reciprocal of (76) we have

$$\frac{1}{f_1} = \frac{f_2}{4{,}900{,}000\ h} + \frac{23{,}000{,}000\left(\frac{h}{l}\right)^2}{4{,}900{,}000\ h}$$

$$= y_1 + y_2$$

and

$$f_1 = \frac{1}{y_1 + y_2} \qquad (77)$$

Fig. 78 gives values of y_1 for different values of f_2, and values of y_2 for different values of the length in feet, L. The values of y_1 and y_2 can be read off the diagram directly for any value of h, f_2, and L. And then, if the sum of y_1 and y_2 be taken on the lower part of the

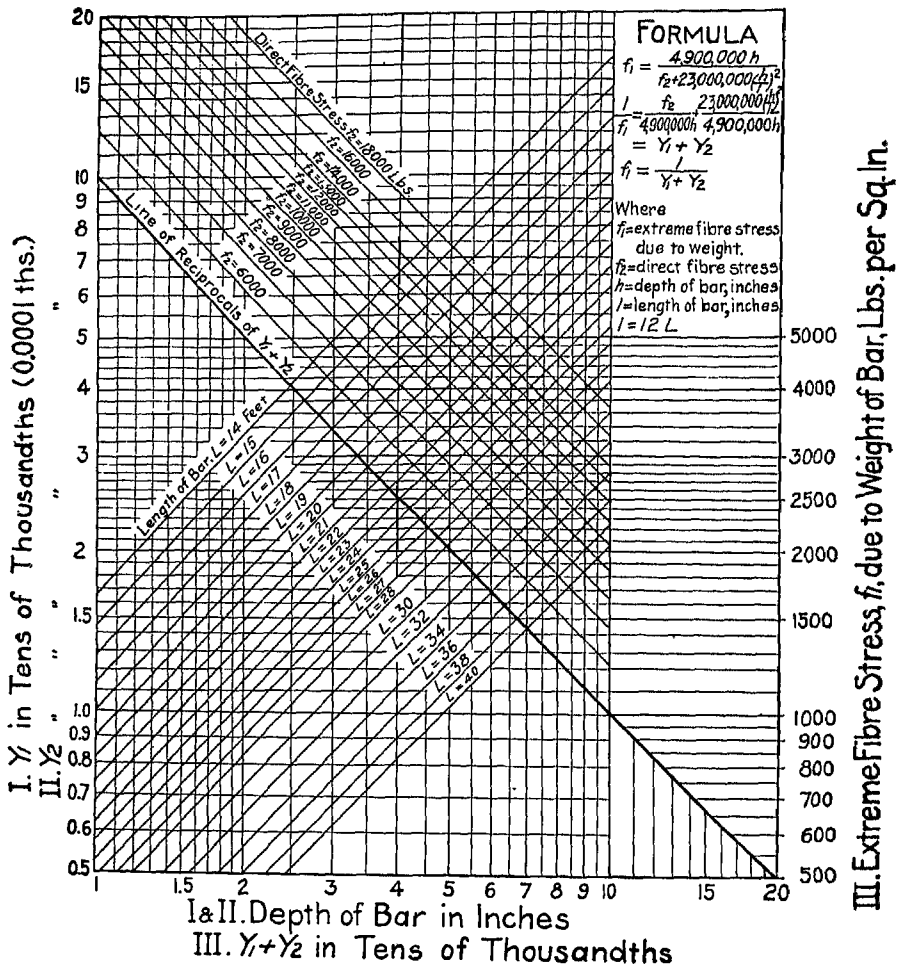


FIG. 78. DIAGRAM FOR FINDING STRESS IN BARS DUE TO THEIR OWN WEIGHT.

diagram, the reciprocal, which is the fibre stress f_1, may be read off the right hand side.

The use of the diagram will be illustrated by two problems:

Problem 1. Required the stress in a $4'' \times 1''$ eye-bar, 20 feet long, which has a direct tension of 56,000 lbs.

In this case, $h = 4''$, $L = 20$ ft., and $f_2 = 14{,}000$ lbs. per sq. in. The stress due to weight, f_1, is found as follows: On the bottom of the diagram find $h = 4$ inches, follow up the vertical line to its intersection with inclined line marked $L = 20$, and then follow the horizontal line passing through the point of intersection out to the left margin and find $y_2 = 3.3$ tens of thousandths; then follow the vertical line $h = 4$ inches, up to its intersection with inclined line marked $f_2 = 14{,}000$, and then follow the horizontal line passing through the point of intersection out to the left margin and find $y_1 = 7.2$ tens of thousandths.

Now to find the reciprocal of $y_1 + y_2 = 7.2 + 3.3 = 10.5$, find value of $y_1 + y_2 = 10.5$ on lower edge of diagram, follow vertical line to its intersection with inclined line marked "Line of Reciprocals" and find stress f_1 by following horizontal line to right hand margin to be

$$f_1 = 950 \text{ lbs. per sq. in.}$$

By substituting in (76) and solving we get $f_1 = 960$ lbs. per sq. in.

Problem 2. Required the stress in a $5'' \times \frac{3}{4}''$ eye-bar, 30 feet long, which has a direct tension of 60,000 lbs., and is inclined so that it makes an angle of 45° with a vertical line.

In this case, $h = 5''$, $L = 30$ feet, $f_2 = 16{,}000$ lbs., and $\theta = 45^\circ$. From the diagram as in Problem 1, $y_2 = 1.8$ tens of thousandths, and $y_1 = 6.5$ tens of thousandths, and

$$f_1 = \frac{1}{y_1 + y_2} \sin\theta = 1200 \times \sin\theta$$

$$= 850 \text{ lbs. per sq. in.}$$

Relations between h, f_1, f_2 and L. For any values of f_2 and L, f_1 will be a maximum for that value of h which will make $y_1 + y_2$ a min-

imum. This value of h will now be determined. Differentiating equation (76) with reference to f_1 and h, we have after solving for h after placing the first derivation equal to zero

$$h = \frac{l}{7800}\sqrt{f_2} \tag{78}$$

in which h is the depth of bar which will have a maximum fibre stress for any given values of l and f_2.

Now if we substitute the value of h in (78) back in equation (76), we find that f_1 will be a maximum when $y_1 = y_2$.

Now in the diagram the values of y_1 and y_2 for any given values of f_2 and L will be equal for the depth of bar, h, corresponding to the intersection of the f_2 and L lines.

It is therefore seen that every intersection of the inclined f_2 and L lines in the diagram, has for an abscissa a value of h, which will have a maximum fibre stress f_1, for the given values of f_2 and L.

For example, for $L = 30$ feet and $f_2 = 12{,}500$ lbs. we find $h = 8.3$ inches and $f_1 = 1700$ lbs. For the given length L and direct fibre stress f_2, a bar deeper or shallower than 8.3 inches will give a smaller value of f_1 than 1700 lbs.

Eccentric Riveted Connections.—The actual shearing stresses in riveted connections are often very much in excess of the direct shearing stresses. This will be illustrated by the calculation of the shearing stresses in the rivets in the standard connection shown in Fig. 79 and Fig. 80.

The eccentric force, P, may be replaced by a direct force, P, acting through the center of gravity of the rivets and parallel to its original direction, and a couple with a moment $M = P \times 3'' = 60{,}000$ inch-lbs. Each rivet in the connection will then take a direct shear equal to P divided by n, where n is the total number of rivets in the connection, and a shear due to bending moment M.

The shear in any rivet due to moment will vary as the distance,

and the resisting moment exerted by each rivet will vary as the square of the distance of the rivet from the center of gravity of all the rivets.

Now, if a is taken as the resultant shear due to bending moment in a rivet at a unit's distance from the center of gravity, we will have the relation

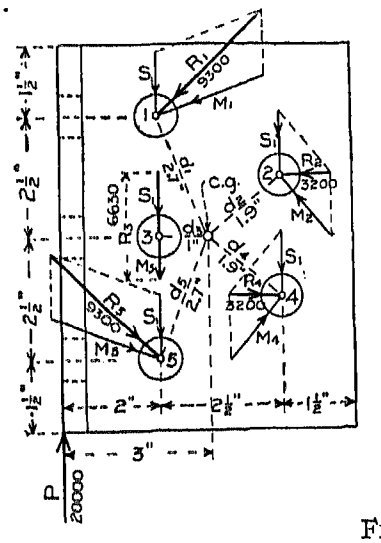
$$M = a\,(d_1^2 + d_2^2 + d_3^2 + d_4^2 + d_5^2)$$

$$= a \Sigma d^2$$

and

$$a = \frac{M}{\Sigma d^2} \tag{79}$$

The remainder of the calculations are shown in Fig. 79. The resultant shears on the rivets are given in the last column of the table



Direct Shear S = 20000 ÷ 5 = 4000 lbs.
Moment = 20000 x 3 = 60000 in. lbs.
= $a(d_1^2 + d_2^2 + d_3^2 + d_4^2 + d_5^2)$
Where a = Moment shear on rivet 3
= 2630 lbs.

Rivet	d	d^2	Moment	M	S	R
1	2.70	7.29	19185	7100	4000	9300
2	1.90	3.61	9500	5000	4000	3200
3	1.00	1.00	2630	2630	4000	6630
4	1.90	3.61	9500	5000	4000	3200
5	2.70	7.29	19185	7100	4000	9300
$a\Sigma d^2 = 22.80\,a = 60000$ in. lbs.				20000		
a = 2630 lbs. = moment shear on rivet 3						

M = Shear due to Moment.
S = Shear due to Direct Load, P
R = Resultant Shear.

0 4000 8000 12000

FIG. 79.

and are much larger than would be expected.

The force and equilibrium polygons for the resultant shears and load P, drawn in Fig. 80, close, which shows that the connection is in equilibrium.

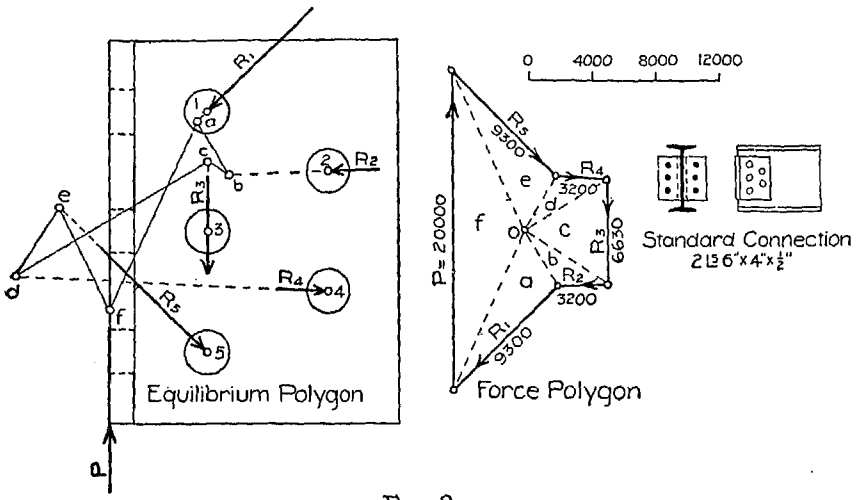


FIG. 80.

STRESSES IN PINS.—A pin under ordinary conditions is a short beam and must be designed (1) for bending, (2) for shear, and (3) for bearing. If a pin becomes bent the distribution of the loads and the calculation of the stresses are very uncertain.

The cross bending stress, S, is found by means of the fundamental formula for flexure, $S = \frac{M c}{I}$, where the maximum bending moment M, is found as explained later; I is the moment of inertia, and c is the radius of a solid or hollow pin.

The safe shearing stresses given in standard specifications are for a uniform distribution of the shear over the entire cross section, and the actual unit shearing stress to be used in designing will be equal to the maximum shear divided by the area of the cross section of the pin.

The bearing stress is found by dividing the stress in the member by the bearing area of the pin, found by multiplying the thickness of the bearing plates by the diameter of the pin.

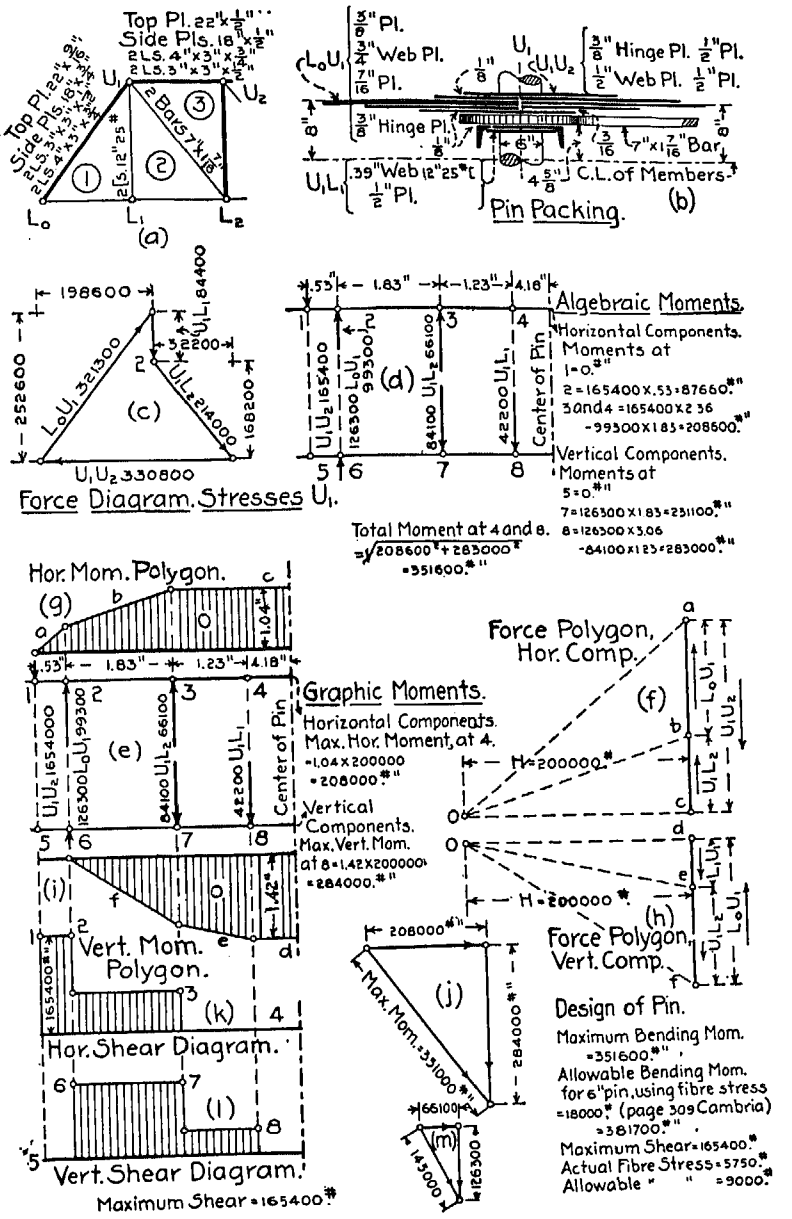


FIG. 80b. CALCULATION OF STRESSES IN A PIN.

Calculation of Stresses.—The method of calculation will be illustrated by calculating the stresses in the pin at U_1 in (a) Fig. 80b. In the complete investigation of the pin U_1, it would be necessary to calculate the stresses when the stress in $U_1 U_2$ was a maximum, and when the stress in $U_1 L_2$ was a maximum. Only the case where the stress in $U_1 U_2$ is a maximum will be considered. However, maximum stresses in pins sometimes occur when the stress in $U_1 L_2$ is a maximum, and this case should be considered in practice.

Bending Moment.—The stresses in the members are shown in (c), which gives the force polygon for the forces. The makeup of the members is shown in (a), and the pin packing on one side is shown in (b). The stresses shown in (c) are applied one-half on each side of the member, the pin acting like a simple beam. The stresses are assumed as applied at the centers of the members.

Algebraic Method.—The amounts of the forces and the distances between their points of application as calculated from (b) are shown in (d). The horizontal and vertical components of the forces are considered separately, the maximum horizontal bending moment and the maximum vertical bending moment are calculated for the same point, and the resultant moment is then found by means of the force triangle.

In (d) the horizontal bending moments are calculated about the points 1, 2, 3, 4; the maximum horizontal moment is to the right of 3, and is 208,600 lb.-in. The vertical bending moments are calculated about points 5, 6, 7, 8; the maximum vertical bending moment is to the right of 8, and is 283,000 lb.-in. The maximum bending moment is at and to the right of 4 and 8, and is $\sqrt{208{,}600^2 + 283{,}000^2} = 351{,}600$ lb.-in.

Substituting in the formula $S = \frac{M c}{I}$, the maximum bending stress is $S = 16{,}600$ lbs. The allowable bending stress for which this bridge was designed was 18,000 lbs.

Graphic Method.—The amounts of the forces and the distances between their points of application are shown in (e). The force polygon for the horizontal components is given in (f), and the bending moment polygon is given in (g). The maximum horizontal bending

moment is to the right of 3, and is $H \times y = 200{,}000 \times 1.04 = 208{,}000$ lb.-in. The force polygon for the vertical forces is given in (h) and the bending moment polygon is given in (i). The maximum vertical bending moment is to the right of 8, and is $H \times y = 200{,}000 \times 1.42 = 284{,}000$ lb.-in. The maximum bending moment occurs at and to the right of 4 and 8, and is 351,000 lb.-in., as shown in (j).

Shear.—The shear is found for both the horizontal and vertical components as in a simple beam, and is equal to the summation of all the forces to the left of the section. The horizontal shear diagram is shown in (k), and the vertical shear diagram is shown in (l). The maximum horizontal shear is between 1 and 2, and is 165,400 lbs. The shear between 2 and 3 is $165{,}400 - 99{,}300 = 66{,}100$ lbs. The maximum vertical shear is between 6 and 7, and is 126,300 lbs. The resultant shear between 2 and 3, and 6 and 7, is $\sqrt{126{,}300^2 + 66{,}100^2} = 145{,}000$ lbs. as in (m), which is less than the horizontal shear between 1 and 2. The maximum shear therefore comes between 1 and 2, and is 165,400 lbs. The maximum shearing unit stress is 5750 lbs. The allowable shearing stress was 9000 lbs.

Bearing.—The bearing stress in $L_0\ U_1$ is $165{,}650 \div 6 \times 1.94 = 14{,}300$ lbs. Bearing stress in $U_1\ U_2$ is $165{,}400 \div 6 \times 1.88 = 14{,}600$ lbs. Bearing stress in $U_1\ L_1$ is $42{,}200 \div 6 \times 0.80 = 7900$ lbs. Bearing stress in $U_1\ L_2$ is $107{,}000 \div 6 \times 1\frac{7}{16} = 12{,}400$ lbs. The allowable bearing stress was 15,000 lbs.

CHAPTER XVa.

Graphic Methods for Calculating the Deflection of Beams.

Introduction.—Algebraic methods for calculating the deflection of beams and the reactions of continuous beams are given in detail in standard works on applied mechanics. The graphic methods are, however, not given the attention that their elegance and simplicity merit. The underlying principles of the graphic method will be developed and a few applications of the method will be given in the

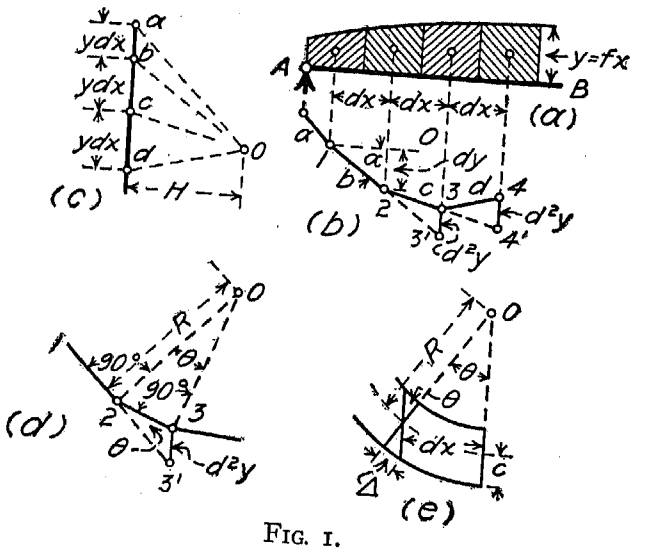


Fig. 1.

following discussion. The discussion will be limited to beams with a constant moment of inertia.

Graphic Equation of the Elastic Curve.—Load a simple beam with a continuous load represented by the equation $y = fx$, as in (a)

Fig. 1. Assume that each differential load, $y\,dx = fx\,dx$, acts through its center of gravity. Now construct a force polygon as in (c) and an equilibrium polygon as in (b) Fig. 1.

Now, in (b) the tangent of the angle between any side of the equilibrium polygon and the X-axis is $\tan a = \frac{dy}{dx}$. If the string $b\,o$ in (b) is produced until it cuts the vertical line through 3, it will cut off the intercept $3\text{–}3^1$, which is the difference between two consecutive values of dy and therefore equals d^2y.

Now, it has been proved that the moment of the force acting through point 2 in (b) about point 3, is equal to the intercept $3\text{–}3^1$ multiplied by the pole distance H, is equal to $3\text{–}3^1 \times H = d^2y\,H$. But the moment of the differential load $fx\,dx$, which acts through point 2, about point 3, is $fx\,dx^2$, and

$$fx\,dx^2 = d^2y\,H$$

and

$$\frac{d^2y}{dx^2} = \frac{fx}{H} \tag{1}$$

It is evident that (1) is the differential equation of the equilibrium polygon in (b).

Now, if the loading is taken so that $y = fx = M$, where M represents the bending moment at any given point x, due to a given loading, the equation for the equilibrium polygon becomes

$$\frac{d^2y}{dx^2} = \frac{M}{H} \tag{2}$$

From mechanics we have the relation that

$$\frac{d^2y}{dx^2} = \frac{M}{EI} \tag{3}$$

which differs from (2) only in having EI substituted for H, E being the modulus of elasticity and I the moment of inertia of the given beam.

This relation may be deduced as follows: In (d) Fig. 1, let equilibrium polygon 1–2–3 represent the neutral axis of a beam as in

(a), the points 1, 2 and 3 being at a distance dx apart. The distortion will be assumed to be so small that $d\,l = dx$. Now, the triangle 2–O–3 may be taken as similar to triangle 3–2–3^1 for small distortions, and

$$O\text{–}2 : 2\text{–}3 :: 2\text{–}3 : 3\text{–}3^1$$

but O–2 equals R, 2–3 equals dx, and 3–3^1 equals d^2y; and, therefore,

$$\frac{d^2y}{dx^2} = \frac{1}{R} \tag{4}$$

Now, in a beam as in (e) Fig. 1, the stresses at any point in the beam will vary as the distance from the neutral axis, and from similar triangles we have

$$R : dx :: c : \Delta$$

and

$$R\,\Delta = c\,dx \tag{5}$$

Now, if S is the fiber stress on the extreme fiber, and E is the modulus of elasticity, we have

$$\Delta : S :: dx : E$$

$$\Delta\,E = S\,dx \tag{6}$$

and, solving (5) and (6) for R, we have

$$R\,S = E\,c$$

But from the common theory of flexure we have $M\,c = S\,I$, and substituting

$$R = \frac{E\,I}{M} \tag{7}$$

Substituting the value of R in (7) and (4) we have

$$\frac{d^2y}{dx^2} = \frac{M}{EI} \tag{3}$$

The preceding discussion gives the following simple graphic method for constructing the elastic curve of a beam:

Construct the bending moment polygon for the given loading on the beam. Load the beam with this bending moment polygon, and

with a force polygon having a pole distance equal to E I, construct an equilibrium polygon; this polygon will be the elastic curve of the beam. It is not commonly convenient to use a pole distance equal to $E I$, and a pole distance H is used, where $N H$ equals $E I$; the deflection at any point will then be equal to the measured ordinate divided by N.

Simple Beam.—The simple beam will be considered when loaded with concentrated and uniform loads, using both algebraic and graphic methods.

Algebraic Method—Concentrated Load at Center of Beam.—The simple beam in (a) Fig. 2, is loaded with a load P at the center. The bending moment diagram is shown in (b) and the beam is loaded with the bending moment diagram in (c) Fig. 2.

To find the equation of the elastic curve take moments of the forces to the left of a point at a distance x from the left support, and

$$-E I y = \frac{P L^2 x}{16} - \frac{P x^3}{12}$$

and

$$48 E I y = P (4 x^3 - 3 L^2 x) \qquad (8)$$

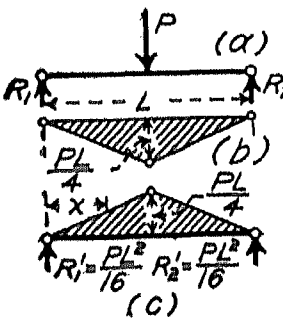


FIG. 2.

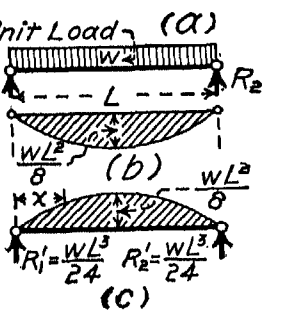


FIG. 3.

The maximum deflection will occur when $x = \frac{1}{2} L$ in (8), or it may be found by taking moments of forces to left of $x = \frac{1}{2} L$ to be

$$\Delta = \frac{P L^3}{48 E I} \qquad (9)$$

Beam Uniformly Loaded.—The simple beam in (a) Fig. 3, is

loaded with a uniform load of w per lineal foot. The bending moment parabola is shown in (b), and the beam is loaded with the bending moment parabola in (c) Fig. 3. To find the equation of the elastic curve, take moments of forces to the left of a point at a distance x from the left support.

The equation of the bending moment parabola with the origin of co-ordinates at the left support is $y = \frac{1}{2} w L x - \frac{1}{2} w x^2$, the area of a segment of the parabola is $A = \frac{1}{4} w L x^2 - 1\text{-}6\, w x^3$, and the center of gravity measured back from x is

$$-X = \frac{x(2L - x)}{6L - 4x}$$

Taking moments of forces to the left of a point x, and reducing, we have

$$24 E I y = w(-x^4 + 2 L x^3 - L^3 x) \qquad (10)$$

The deflection is a maximum when $x = \frac{1}{2} L$, and may be found directly by taking moments, or may be found from (10), and is

$$\Delta = \frac{5}{384} \frac{w L^4}{E I} \qquad (11)$$

Cantilever Beam—*Concentrated Load.*—The cantilever beam in (a) Fig. 4, has a concentrated load, P, at its extreme end. It will be seen that the cantilever beam may be considered as one-half of a simple beam with a span $2L$, and a load $2P$, at the center. The equation of the elastic curve may be found as in Fig. 2. Load the beam with the bending moment diagram as in (b) Fig. 4, and considering the cantilever as one-half of the simple beam we have, after reducing,

$$6 E I y = 3 P L^2 x - P x^3 \qquad (12)$$

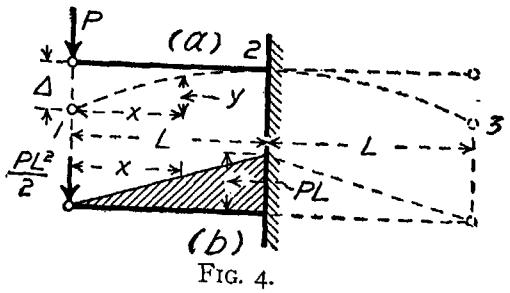


FIG. 4.

The maximum value of Δ is equal to y when x equals L, and

$$\Delta = \frac{P L^3}{3 E I} \tag{13}$$

To find the maximum deflection we may take the moment of the entire bending-moment parabola about the point 1, and

$$E I \Delta = \frac{P L^2}{2} \times \frac{2}{3} L, \text{ and}$$

$$\Delta = \frac{P L^3}{3 E I}$$

This method of finding the maximum deflection of a cantilever beam is the one to use in calculations, and will be used in the solution of the problem of the transverse bent.

Simple Beam—*Graphic Method.*—In Fig. 5 a simple beam is loaded with a load P, as shown. With force polygon (b), draw equilibrium polygon (c). Now load the beam with equilibrium polygon as in (c), and divide the area of the equilibrium polygon into segments, which are treated as loads acting through their centers of gravity. Construct force polygon (d) and draw equilibrium polygon (e).

Now, the deflection at any point having an ordinate y in (e) will be, if proper scales are used,

$$\Delta = \frac{y \times H \times H^1}{E I}$$

In Fig. 5, if P equals 3000 lbs., and the area of the equilibrium polygon and pole distance H^1 are measured in square-foot pounds, pole distance H in pounds, and y in feet, we will have

$$\Delta = \frac{y \times H \times H^1 \times 1728}{E I}$$

= 1.88 inches at center, while maximum value of deflection is Δ^1 = 1.92 inches.

Tangents to Elastic Curve.—If strings 1 and 3 in (e) be pro-

duced, they will intersect at 2 on a line through the center of gravity of the moment-area polygon, and the strings 1–2 and 2–3 will be tangents to the elastic curve at the supports R_1 and R_2, respectively. This gives an easy method of constructing the tangents to the elastic curve without constructing the curve. It is also seen that the tan-

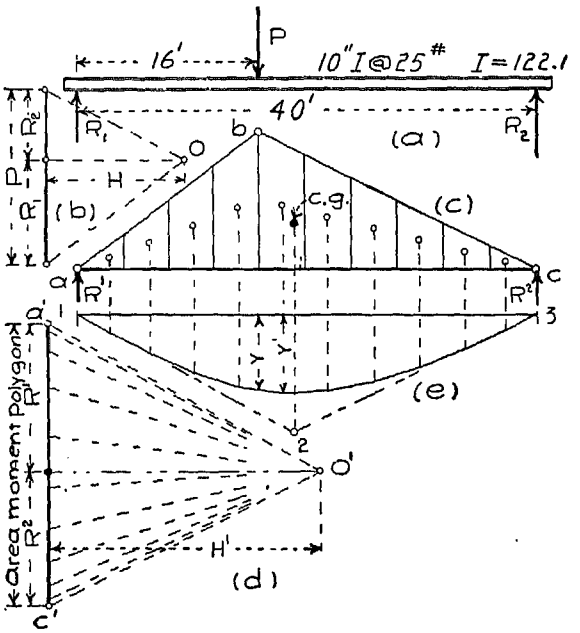


FIG. 5.

gents to the elastic curve depend only on the amount of the moment area and position of its center of gravity, and are independent of the arrangement of the moment areas.

Continuous Beams.—A beam which in an unstrained condition rests on more than two supports is a continuous beam. For a straight beam the supports must all be on the same level. Beams of one span with one end fixed and the other end supported, or with both ends fixed, may also be considered as continuous beams.

In Fig. 6a the continuous beam in (a) with spans L_1 and L_2 carries a uniform load w per lineal foot. It is required to calculate the reactions R_1, R_2, and R_3.

The reactions of the continuous beam in (a) may be replaced by the reactions of the two simple beams loaded with the uniform load w in (b), and the reactions and the load of the simple beam with the span $L_1 + L_2$ and carrying a negative load r_2' in (c). The reactions in (a) will then have the following values; $R_1 = r_1 - r_1'$; $R_2 = r_2 + r_2'$; $R_3 = r_3 - r_3'$.

Now the upward curvature of the beam in (a) due to the load r_2' will be neutralized by the load above equal to r_2' which is transferred to the reaction R_2 by flexure in the beam. The upward deflection of

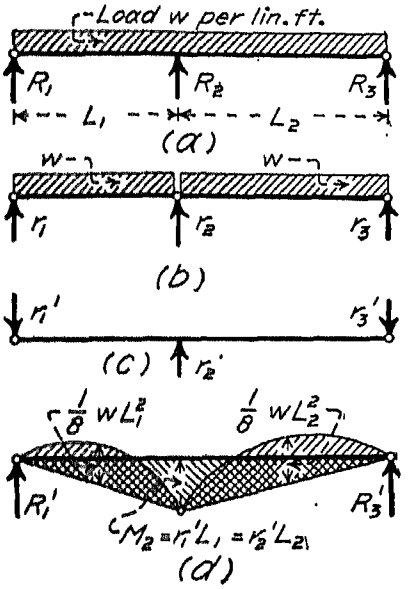


FIG. 6a.

the beam in (c) at any point will be the bending moment divided by $E\,I$ at the same point in (d) due to a bending moment polygon with a maximum moment $M_2 = r_1' \times L_1 = r_3' \times L_2$; and the downward deflection of the beam in (b) at any point will be the bending moment divided by $E\,I$ at the same point in (d) due to the bending moment polygons for a uniform load w covering the simple spans in (b). But the deflection of the beam in (a) is zero at the reaction R_2, and therefore the bending moment at the corresponding point in (d) is zero.

From the above discussion it follows that to calculate the reactions of the continuous beam in (a) by moment areas, take a simple beam with a span equal to $L_1 + L_2$, and load it with the bending moment polygons for beams (b) and (c) as in (d); the bending moment in beam (d) at the points corresponding to the reactions will be equal to zero, and the reactions of beam (a) can be calculated by statics when the M_2 is obtained.

Continuous Beam—Concentrated Loads.—In (a) Fig. 6, a continuous beam of two equal spans of length L, is loaded with two equal loads P, at the centers of the spans. Calculate the bending

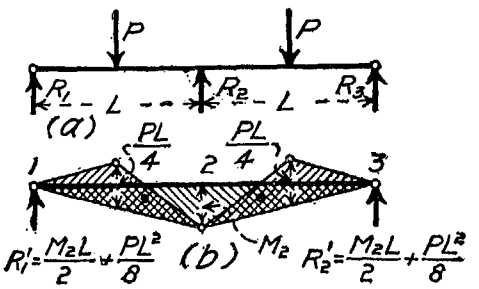


FIG. 6.

moments and load a simple beam with a span equal to $2L$, with the bending-moment diagrams due to P in each span, and with the negative bending-moment diagram due to the reaction R_2. Then to find M_2, the bending moment at 2, take moments of forces to the left of 2, and

$$\frac{M_2 L^2}{2} + \frac{P L^3}{8} - \frac{M_2 L^2}{6} - \frac{P L^3}{16} = 0$$

$$M_2 = -\frac{3}{16} P L$$

To calculate R_1 take moments in (a) about 2, and

$$R_1 L - \frac{P L}{2} - M_2 = 0$$

Beam Uniformly Loaded.—In (a) Fig. 7, a continuous beam of three equal spans of length L, is loaded with a uniform load equal to w per foot. Calculate the bending moments due to a uniform load of w on each span, and load a simple beam of span $3L$ with the positive bending-moment diagrams due to load w, and with the negative bending-moment diagrams due to the reactions R_2 and R_3. The bending moment M_2 is equal to M_3. Now the deflection of the beam is zero at 2 and 3, and the bending moments must, therefore, be zero at these points. Taking moments of forces to the left of 2, we have

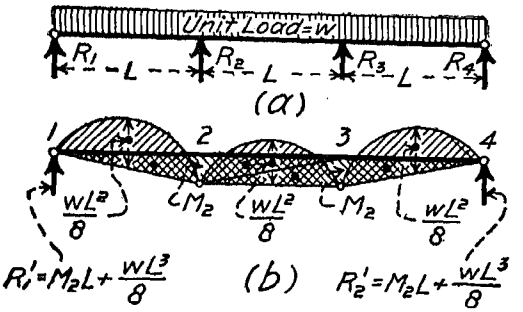


FIG. 7.

$$M_2 L^2 + \frac{w L^4}{8} - \frac{w L^4}{24} - \frac{M_2 L^2}{6} = 0$$

$$M_2 = -\frac{w L^2}{10} = M_3$$

To calculate R_1 take moments about 2 in (a), and

$$R_1 L - \frac{w L^2}{2} - M_2 = 0$$

$$R_1 = \frac{4}{10}\, wL = R_4;$$

$$R_2 = R_3 = \frac{3}{2} w L - \frac{4}{10} w L = \frac{11}{10} w L$$

Continuous Beam of n Spans.—To calculate the reactions for a continuous beam of n spans, equal or unequal, loaded with any system or systems of loads proceed as follows:

Calculate the bending moment due to the external load, or loads, or system of loads in each span considered as a simple beam. Take a simple beam having a total length equal to the length of the continuous beam, and load it with the bending moment polygons found as above. Also load the beam with the bending moment polygons due to the reactions. The reactions being unknown, the bending moments at the reactions will be unknown. Now calculate the bending moment in the simple beam at points corresponding to each reaction and place the result equal to zero, for the reason that the deflection at the supports is zero.

For a continuous beam of n spans there will be $n + 1$ equations which is equal to the number of unknown reactions. Solving these equations the unknown moments will be found, and the reactions may be calculated algebraically.

Transverse Bent.—The problem of the calculation of the point of contra-flexure in the columns of a transverse bent—the algebraic

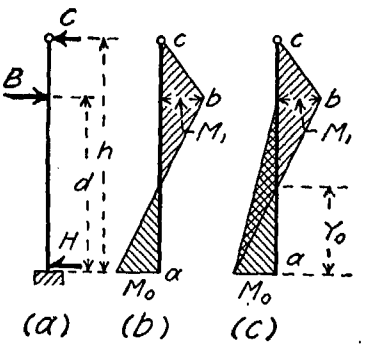


FIG. 8.

solution of which is given in Chapter XI—will now be solved by the use of moment areas. The nomenclature in Fig. 8 is the same as in

Chapter XI. It is assumed that the deflection at points b and c are equal.

In (b) Fig. 8, the deflection at b from the tangent at a is found by taking moments of the moment areas below b to be

$$E\,I\,\Delta = \frac{M_0\,d}{2}\,\frac{2}{3}\,d - \frac{M_1\,d}{2}\,\frac{d}{3}$$

$$\Delta = \frac{2\,M_0\,d^2 - M_1\,d^2}{6\,E\,I} \qquad (14)$$

The deflection at c from the tangent at a is found by taking moments of moment areas below c to be

$$E\,I\,\Delta^1 = \frac{M_0\,d}{2}\,(h - d/3) - \frac{2\,M_1\,(h-d)^2}{6} - \frac{M_1\,d}{2}(h - 2/3\,d)$$

$$\Delta^1 = \frac{M_0\,(3\,d\,h - d^2) - M_1\,(2\,h^2 - h\,d)}{6\,E\,I} \qquad (15)$$

But Δ is equal to Δ^1 by hypothesis, and equating (14) and (15) we have

$$2\,M_0\,d^2 - M_1\,d^2 = M_0\,(3\,d\,h - d^2) - M_1\,(2\,h^2 - h\,d)$$

transposing,

$$M_0\,(3\,h\,d - 3\,d^2) = M_1\,(2\,h^2 - h\,d - d^2) \qquad (16)$$

Now in (c) Fig. 8, it will be seen that $M_0 : M_1 :: y_0 : d - y_0$, and

$$M_0\,(d - y_0) = M_1\,y_0 \qquad (17)$$

Solving (16) and (17) for y_0, we have

$$y_0 = \frac{d}{2}\,\frac{(2\,h + d)}{(h + 2\,d)} \qquad (18)$$

which is the same value as was found by algebraic methods.

Reactions of Simple Draw Bridges.—The preceding methods are not adapted to the solution of problems involving moving loads, as in draw bridges. The following method, which is an application of curved influence lines, is quite simple in theory and application, al-

though requiring considerable labor in preparing the diagrams. The solution will first be explained and a proof given later:

Draw Bridge With Three Supports.—In Fig. 9 a continuous beam with spans L_1 and L_2 is loaded with concentrated moving loads represented by P_1 and P_2, as in (a).

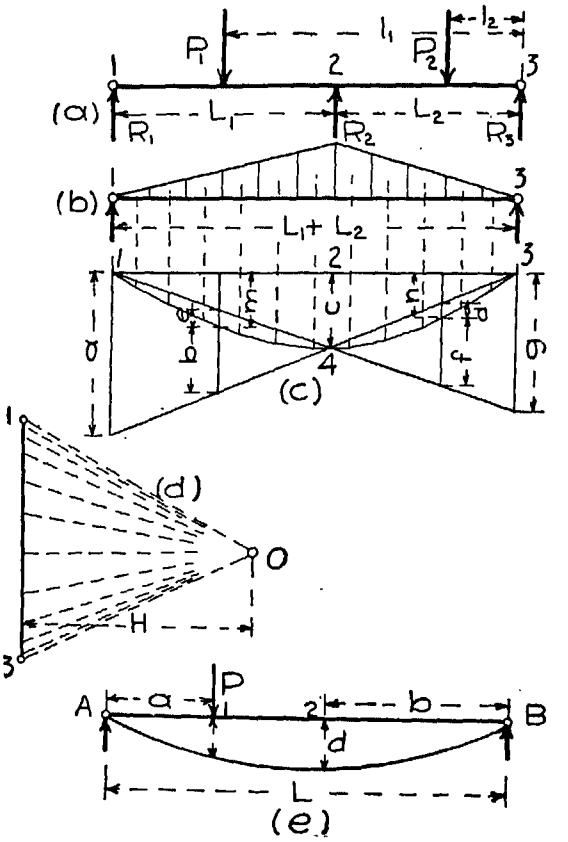


FIG. 9.

In (b) load a simple beam having a span $L_1 + L_2$ with a bending-moment polygon due to the reaction R_2 (the value of R_2 is unknown, and any convenient load will do).

Divide the bending-moment diagram into segments, construct a force polygon as in (d) and draw an equilibrium polygon as in (c)

Fig. 9, assuming that the segments are loads acting through their centers of gravity.

The pole distance H may be taken as any convenient length, and the pole O may be taken at any point [in (c) the pole has been selected to bring the closing line horizontal for convenience only].

Then in (c)

$$R_1 a = P_1 b - P_2 d$$

and

$$R_1 = \frac{P_1 b - P_2 d}{a} \tag{19}$$

$$R_3 c = P_1 m + P_2 n$$

and

$$R_3 = \frac{P_1 m + P_2 n}{c} \tag{20}$$

$$R_3 g = -P_1 e + P_2 f$$

and

$$R_3 = \frac{-P_1 e + P_2 f}{g} \tag{21}$$

Proof.—The ordinates to the equilibrium polygon in (c) are proportional to the ordinates to the true elastic curve of the beam in (b) when it is loaded with a given load at 2.

Now in (c) Fig. 9, if the deflection at 2 due to a load P at 1 is d, then if the load P be moved to 2, the deflection at 1 will be d. This can be proved by calculating the bending moments at 2 and 1 for the conditions, since the deflections are directly proportional to the bending moments. With P at 1, the bending moment at 2 is $\frac{P a b}{L}$; and with P at 2, the bending moment at 1 is $\frac{P b a}{L}$, and the proposition is proved.

Now in (c), if the deflection due to a load unity at 2 is m at P_1, then the deflection at 2 due to a load unity at P_1 will be m. If load R_3 is applied at 2, the work done in making the elastic curve pass through 2 will be $R_3 c$; while the resistance due to a load P_1 will be P_1 times the deflection at 2 due to the load P_1, which is equal to $P_1 m$. In like manner the resistance due to P_2 will be $P_2 n$, and

$$R_2 c = P_1 m + P_2 n$$

and

$$R_2 = \frac{P_1 m + P_2 n}{c} \tag{20}$$

To find R_1 take moments about 3 in (a), and

$$R_1 (L_1 + L_2) + R_2 L_2 - P_1 l_1 - P_2 l_2 = 0$$

and from similar triangles in (c)

$$R_1 a + R_2 c - P_1 (m + b) - P_2 (n - d) = 0 \tag{22}$$

Substituting the value of R_2 from (20) in (22), we have

$$R_1 a = P_1 b - P_2 d$$

$$R_1 = \frac{P_1 b - P_2 d}{a} \tag{19}$$

and since

$$R_1 + R_2 + R_3 = P_1 + P_2$$

$$R_3 = \frac{-P_1 e + P_2 f}{g} \tag{21}$$

Uniform Load.—For a uniform load on the beam the areas of the diagram covered by the uniform load will be used in the place of the ordinates as in Fig. 9 (see discussion on Influence Diagrams, Chapter X). For example in Fig. 10 the reactions are given by the following formulas:

$$R_1 = \frac{p \text{ (area } B_1 - \text{area } B_2)}{a} \tag{23}$$

$$R_2 = \frac{p \text{ (area } A + \text{area } B_2 + \text{area } C_1)}{c} \tag{24}$$

$$R_3 = \frac{p \text{ (area } C_2 - \text{area } C_1)}{g} \tag{25}$$

Draw Bridge with Four Supports.—To find the reaction at R_2 in Fig. 11, proceed as follows: With a load represented by the triangle 1–2–4, construct a force polygon (not shown) and draw an equi-

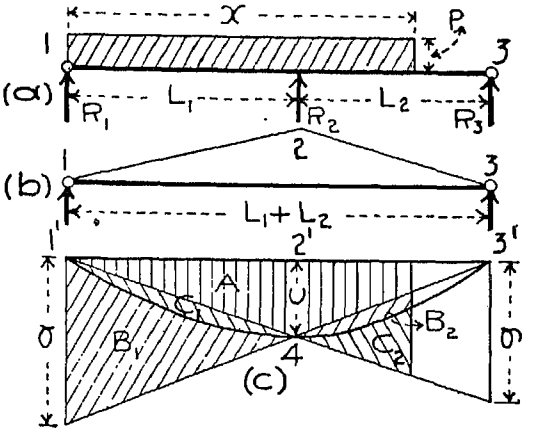


FIG. 10.

librium polygon passing through *m–n–o–p*. Now, with a load represented by the triangle 1–3–4, construct a force polygon (not shown) and draw an equilibrium polygon passing through *m–o–p*. The method of drawing an equilibrium polygon through three points is explained in Chapter V, Fig. 20.

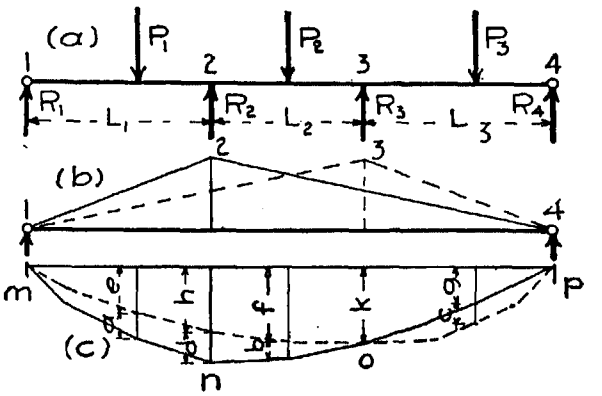


FIG. 11.

Then in (c) Fig. 11,

$$R_2 d = P_1 a + P_2 b - P_3 c$$

and

$$R_2 = \frac{P_1 a + P_2 b - P_3 c}{d} \tag{26}$$

R_3 may be found in a similar manner by drawing an equilibrium polygon for a load, 1–3–4, through point n.

When R_2 and R_3 have been obtained, the reactions R_1 and R_4 can most easily be obtained by algebraic moments.

Proof.—With the load, 1–2–4, and full line deflection curve we have, as in the case of three supports,

$$R_2 (d + h) = -R_3 k + P_1 (a + e) + P_2 (b + f) + P_3 g \tag{27}$$

And with the load, 1–3–4, and dotted line deflection curve we have, in like manner,

$$R_3 k = -R_2 h + P_1 e + P_2 f + P_3 (c + g) \tag{28}$$

Subtracting (28) from (27) we have

$$R_2 d = P_1 a + P_2 b - P_3 c$$

$$R_2 = \frac{P_1 a + P_2 b - P_3 c}{d} \tag{29}$$

which is the equation of which proof was required.

PART III.

DESIGN OF MILL BUILDINGS.

CHAPTER XVI.

General Design.

General Principles.—The general dimensions and outline of a mill building will be governed by local conditions and requirements. The questions of light, heat, ventilation, foundations for machinery, handling of materials, future extensions, first cost and cost of maintenance should receive proper attention in designing the different classes of structures. One or two of the above items often determines the type and general design of the structure. Where real estate is high, the first cost, including the cost of both land and structure, causes the adoption in many cases of the multiple story building, while on the other hand where the site is not too expensive the single story shop or mill is usually preferred. In coal tipples and shaft houses the handling of materials is the prime object; in railway shops and factories turning out heavy machinery or a similar product, foundations for the machinery required, and convenience in handling materials are most important; while in many other classes of structures such as weaving sheds, textile mills, and factories which turn out a less bulky product with light machinery, and which employ a large number of men, the principal items to be considered in designing are light, heat, ventilation and ease of superintendence.

Shops and factories are preferably located where transportation facilities are good, land is cheap and labor plentiful. Too much care cannot be used in the design of shops and factories for the reason that defects in design that cause inconvenience in handling materials and workmen, increased cost of operation and maintenance are permanent and cannot be removed.

The best modern practice inclines toward single floor shops with as few dividing walls and partitions as possible. The advantages of this type over multiple story buildings are (1) the light is better, (2) ventilation is better, (3) buildings are more easily heated, (4) foundations for machinery are cheaper, (5) machinery being set directly on the ground causes no vibrations in the building, (6) floors are cheaper, (7) workmen are more directly under the eye of the superintendent, (8) materials are more easily and cheaply handled, (9) buildings admit of indefinite extension in any direction, (10) the cost of construction is less, and (11) there is less danger from damage due to fire.

The walls of shops and factories are made (1) of brick, stone, or concrete; (2) of brick, hollow tile or concrete curtain walls between steel columns; (3) of expanded metal and plaster curtain walls and glass; (4) of concrete slabs fastened to the steel frame; and (5) of corrugated iron fastened to the steel frame.

The roof is commonly supported by steel trusses and framework, and the roofing may be slate, tile, tar and gravel or other composition, tin or sheet steel, laid on board sheathing or on concrete slabs, tile or slate supported directly on the purlins, or corrugated steel supported on board sheathing or directly on the purlins. Where the slope of the roof is flat a first grade tar and gravel roof, or some one of the patent composition roofs is used in preference to tin, and on a steep slope slate is commonly used in preference to tin or tile. Corrugated steel roofing is much used on boiler houses, smelters, forge shops, coal tipples, and similar structures.

Floors in boiler houses, forge shops and in similar structures are generally made of cinders; in round houses brick floors on a gravel or

concrete foundation are quite common; while in buildings where men have to work at machines the favorite floor is a wooden floor on a foundation of cinders, gravel, or tar concrete. Where concrete is used for the foundation of a wooden floor it should be either a tar or an asphalt concrete, or a layer of tar should be put on top of the cement concrete to prevent decay. Concrete or cement floors are used in many cases with good results, but they are not satisfactory where men have to stand at benches or machines. Wooden racks on cement floors remove the above objection somewhat. Where rough work is done, the upper or wearing surface of wooden floors is often made of yellow pine or oak plank, while in the better classes of structures, the top layer is commonly made of maple. For upper floors some one of the common types of fireproof floors, or as is more common a heavy plank floor supported on beams may be used.

Care should be used to obtain an ample amount of light in buildings in which men are to work. It is now the common practice to make as much of the roof and side walls of a transparent or translucent material as practicable; in many cases fifty per cent of the roof surface is made of glass, while skylights equal to twenty-five to thirty per cent of the roof surface are very common. Direct sunlight causes a glare, and is also objectionable in the summer on account of the heat. Where windows and skylights are directly exposed to the sunlight they may best be curtained with white muslin cloth which admits much of the light and shades perfectly. The "saw tooth" type of roof with the shorter and glazed tooth facing the north, gives the best light and is now coming into quite general use.

Plane glass, wire glass, factory ribbed glass, and translucent fabric are used for glazing windows and skylights. Factory ribbed glass should be placed with the ribs vertical for the reason that with the ribs horizontal, the glass emits a glare which is very trying on the eyes of the workmen. Wire netting should always be stretched under skylights to prevent the broken glass from falling down, where wire glass is not used.

Heating in large buildings is generally done by the hot blast system in which fans draw the air across heated coils, which are heated by exhaust steam, and the heated air is conveyed by ducts suspended from the roof or placed under the ground. In smaller buildings, direct radiation from steam or hot water pipes is commonly used.

The proper unit stresses, minimum size of sections and thickness of metal will depend upon whether the building is to be permanent or temporary, and upon whether or not the metal is liable to be subjected to the action of corrosive gases. For permanent buildings the author would recommend 16,000 lbs. per square inch for allowable tensile, and $16{,}000 - 70\frac{l}{r}$ lbs. per square inch for allowable compressive stress for direct dead, snow and wind stresses in trusses and columns; l being the center to center length and r the radius of gyration of the member, both in inches. For wind bracing and flexural stresses in columns due to wind, add 25 per cent to the allowable stresses for dead, snow and wind loads. For temporary structures the above allowable stresses may be increased 20 to 25 per cent.

The minimum size of angles should be $2'' \times 2'' \times \frac{1}{4}''$, and the minimum thickness of plates $\frac{1}{4}''$, for both permanent and temporary structures. Where the metal will be subjected to corrosive gases as in smelters and train sheds, the allowable stresses should be decreased 20 to 25 per cent, and the minimum thickness of metal increased 25 per cent, unless the metal is fully protected by an acid-proof coating (at present the best paints do little more in any case than delay and retard the corrosion).

The minimum thickness of corrugated steel should be No. 20 gage for the roof and No. 22 for the sides; where there is certain to be no corrosion Nos. 22 and 24 may be used for the roof and sides respectively.

The different parts of mill buildings will be taken up and discussed at some length in the following chapters.

CHAPTER XVII.

FRAMEWORK.

Arrangement.—The common methods of arranging the framework in simple mill buildings are shown in Fig. 1, Fig. 81 and Fig. 82. The different terms which are used in the discussion that follows will be made clear by an inspection of Fig. 1 and Fig. 81.

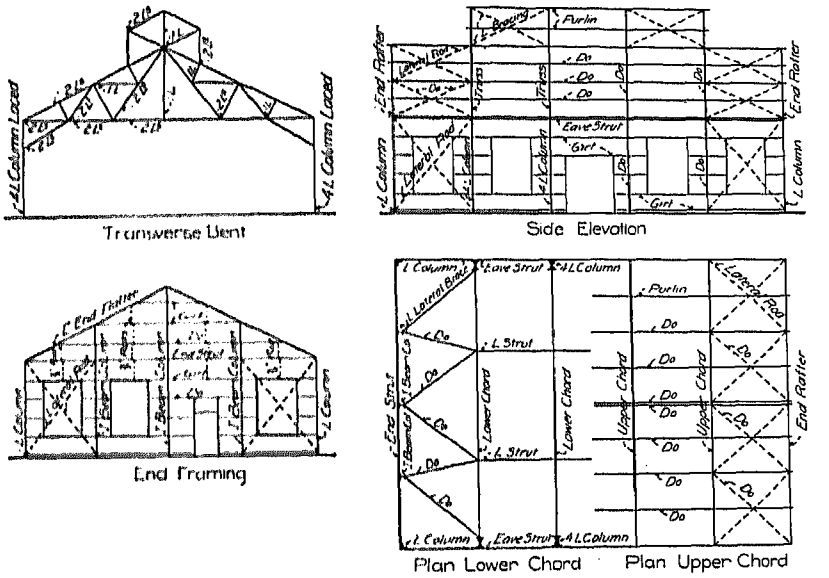


FIG. 81.

The three types of mill buildings—steel frame mill buildings, mill buildings with masonry filled walls, and mill buildings with masonry walls—have been discussed in the Introduction.

The end post bent, shown in (a) Fig. 1 and in Fig. 81, usually requires less material than the end trussed bent shown in (b) Fig. 1 and in Fig.

82, and is commonly used for simple mill buildings. Extensions can be made with about equal ease in either case, and the choice of methods will usually be determined by the local conditions of the problem and

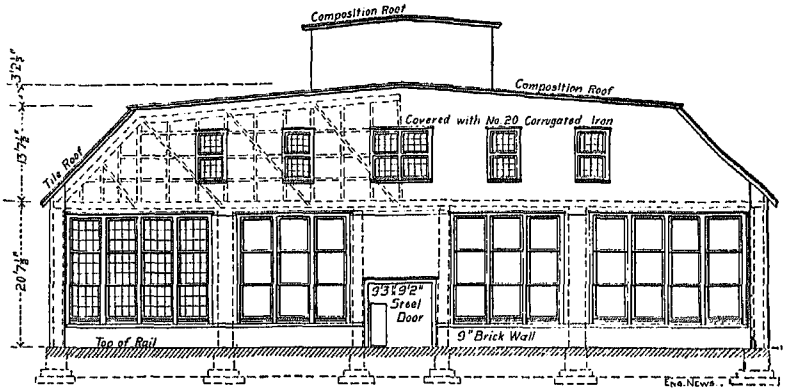


FIG. 82. A. T. & S. F. R. R. BLACKSMITH SHOP, TOPEKA, KAS.

the fancy of the designer. In train sheds and similar structures the end trussed bent (b), Fig. 1, is used. Where the truss span is quite long, as in train sheds, the end trusses are often designed for lighter loads than are the intermediate trusses, thus saving considerable material. In the case of simple mill buildings of moderate size all trusses are, however, commonly made alike, the extra cost of detailing being usually more than the amount saved in material.

In train sheds, coliseums, and similar structures requiring a large floor space, the three-hinged arch is very often used in place of the typical transverse bent system.

The various parts of the framework of mill buildings will be taken up and discussed in order.

TRUSSES.—Types of Trusses.—The proper type of roof truss to use in any particular case will depend upon the span, clear headroom, style of truss preferred, and other conditions. For spans up to about 100 feet, the Fink type of truss is commonly used. This type of truss has the advantage of short struts, simplicity of details and economy. The stresses that control the design are with but a very few exceptions

those caused by an equivalent uniform dead load, thus simplifying the calculation of stresses (see Table VI).

The outline of the truss will depend upon the spacing of the purlins, and upon whether or not the purlins are placed at the panel points of the truss. The most economical and pleasing arrangement is to make a panel point in the truss under each purlin. Taking the normal wind load on the roof at from 25 to 30 lbs. per sq. ft., it will be seen in Fig. 112 that for Nos.20 and 22 corrugated steel, when used without sheathing, the purlins should be spaced from 4 to 5 feet. If this spacing is exceeded corrugated steel roofing supported directly on the purlins is almost certain to leak. Where sheathing is used the purlin spacing can be made greater. Many designers, however, pay no attention to the matter of placing the purlins at the panel points, the upper chord of the truss being stiffened to take the flexural stress.

In Fig. 83, (a) shows the form of a Fink truss for a span of 30 feet; (b) for a span of 40 feet; (c) for a span of 50 feet; (d) for a span of 60 feet; and (e) for a span of 80 feet, on the assumption that the purlins are spaced from 4 to 5 feet, and come at the panel points of the truss. If trusses with vertical posts are desired the triangular trusses (h) and (j), or Fink truss (f) may be used. The truss shown in (i) is occasionally used for long spans, although it has little to recommend it except novelty. The truss shown in (k) is used where there is ample headroom. The quadrangular truss shown in (l) and the camels back truss shown in (m), are used for long spans where the appearance of the truss is an important feature, as in convention halls and train sheds. The lower chords of mill building trusses are usually made horizontal, but by giving the lower chord a camber, as in (g), the appearance from the side is greatly improved.

The "saw tooth" or "weaving shed" roof shown in (a) Fig. 84, has been used abroad for many years and is now coming into quite general use in this country for shops and factories as well as for weaving sheds, as indicated by the name. The short leg of the roof is made inclined as in (a), or vertical as in (b), and is glazed with glass or

translucent fabric. The glazed leg of the roof is made to face the north, thus giving a constant and agreeable light and doing away with the use of window shades.

The principal difficulty in saw tooth roof construction is in obtain-

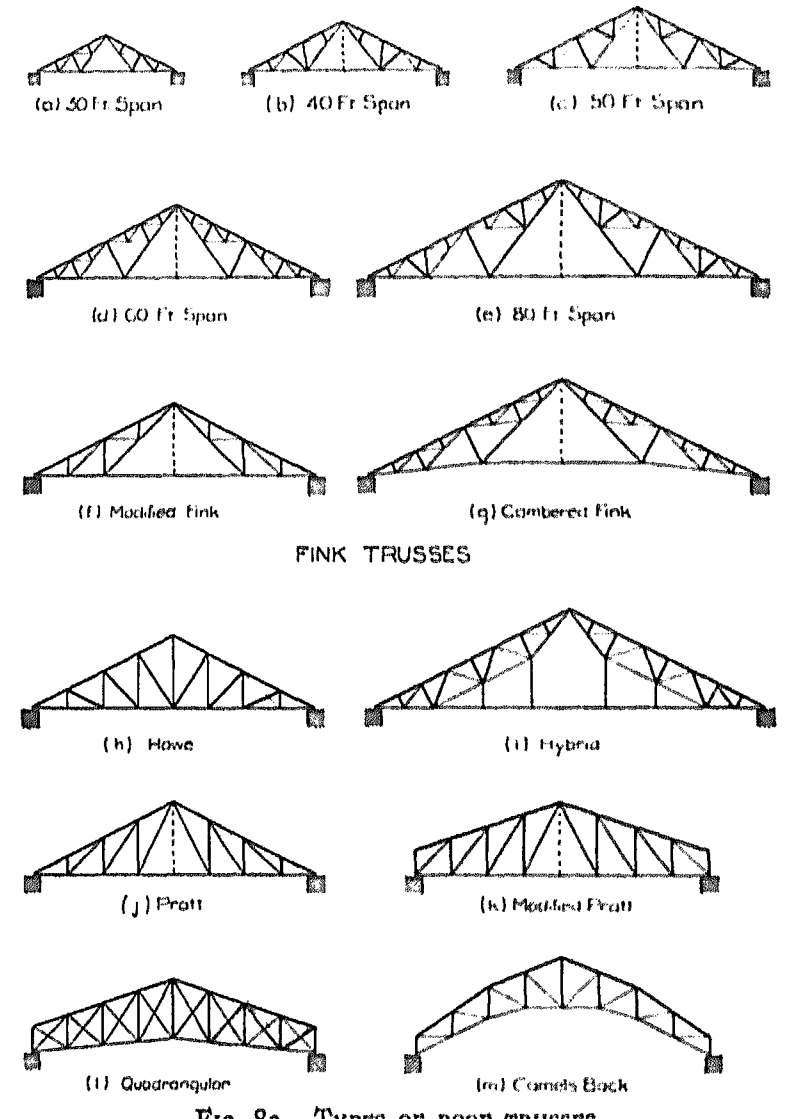
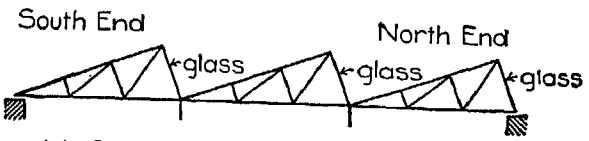


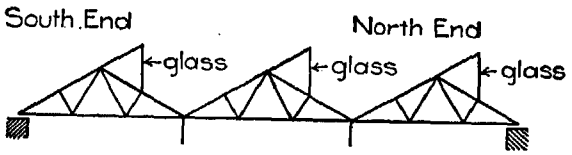
FIG. 83. TYPES OF ROOF TRUSSES.

ing satisfactory and efficient gutters, and in preventing condensation on the inner surface of the glass and gutters. Another objection to the use of saw tooth roofs in localities having a heavy snowfall is that the snow drifts the roof nearly full and shuts off the light. The common method of preventing the snow from collecting, and for taking care of the roof water, is that given in the description of the Conkey plant, which see.

The modified saw tooth roof shown in (b), Fig. 84, is proposed by the author as a substitute for the usual type of saw tooth roof shown in (a). This modified saw tooth roof allows the use of ordinary valley gutters, and gives an opportunity to take care of the condensation on the inner surface of the glass by suspending a gutter at the bottom of the monitor leg. Snow will cause very little trouble



(a) Saw Tooth (Weaving Shed)



(b) Modified Saw Tooth

FIG. 84.

with this roof on account of the increased depth of gutter. The modified saw tooth roof has a greater pitch, and has a more economical truss for long spans than the common form shown in (a). Condensation on the inner surface of the glazed leg can be practically prevented by using double glazing with an air space between the sheets of glass. Double

glazing in windows and skylights makes the building much easier to heat, the air space making an almost perfect non-conductor.

Brown & Sharpe Foundry.—In the Brown & Sharpe Mfg. Company's Foundry, a modification of the saw tooth roof was adopted in which glass was used on both surfaces of the roof. The skylights extend east and west and have a pitch of 45 degrees. The southerly pitch is glazed with opaque glass, the other with ordinary rough glass. The ventilator monitor, which surmounts the skylights, is glazed with opaque glass on the southerly side, and extends high enough so that no light up to an angle of 70 degrees reaches the glass below. By this arrangement no direct sunlight is admitted to the shop from above excepting for a few minutes at noon during the longest days of the year. The result of this overhead light, combined with the almost wholly glass walls of the room is that the floor below is as light as out of doors, to all intents and purposes, yet diffused light only is admitted. A rod placed upright on the floor of one of these rooms casts no shadow.

*Conkey Printing Plant**.—The printing plant of the W. B. Conkey Co., Hammond, Ind., consists of a single story building, 540 × 450 ft. The roof is of the weaving shed or saw tooth type and all windows are glazed with frosted glass and are placed at an angle, looking toward the north. Every 29 feet of roof space provides 11 feet of light. Owing to the angle of the roof the direct rays of light are kept out of the building, which is thus lighted by the soft reflected rays from the northern sky. The entire roof is built up out of light structural steelwork resting on cast iron columns spaced 29 ft. c. to c. one way, and 16 ft. c. to c. in the other direction. The height of the trusses above the floor is 12 ft. To prevent the snow collecting in the valleys between the skylights, the bottom of the gutter and the glass are kept heated so that the snow melts as it falls. This method produces condensation on the inner surface of the glass, which is collected in a system of condensation gutters and carried outside the building.

The heating and ventilating of the building is accomplished by a blast system, with the heating ducts under the floor, which supply registers throughout the plant, arranged on the side walls of each department. The heating system can be made to produce a mild heat for the seasons of spring and fall, and can also be turned into a cooling system in the summer, by running cold water through the steam pipes at

*Engineering News, Dec. 8, 1898.

the fan and changing the air every 15 minutes with cool air in hot weather.

The floor is built of heavy plank and finished maple laid on sleepers which are bedded in cinders. The walls are made of heavy tile and the openings are closed with iron fire doors. The building is practically fireproof and takes a very low rate of insurance.

Boyer Plant.—The Boyer Plant of the Chicago Pneumatic Tool Co., at Detroit, Mich., is 325 × 185 ft., with the longer dimension extending north and south. The roof of the building is divided into two sections, having spans of about 92 ft. each, a pitch of about ¼, and is covered with Patent Asbestos Roofing—manufactured by H. W. Johns-Manville Co., Milwaukee, Wis.—laid on 1⅛-in. plank sheathing. The building is lighted by means of saw tooth skylights facing north and extending from the ridge of the roof to within about 6 ft. of the eaves on the outside and the valley gutter on the inside. The trusses are spaced 16 ft. apart, and there are three saw tooth skylights between each pair of trusses, making 240 skylights in the roof. The north leg of the saw tooth is vertical and is glazed with double corrugated glass, the south leg is covered with asbestos roofing. The building is ventilated by means of circular ventilators placed in the ridge of the roof and spaced 16 ft. apart. The lighting in this building is almost perfect. The roofing has given satisfaction with the exception of the large valley gutters, which will be covered with copper or lead in the near future. There has been a little trouble with condensation, but not enough to make it necessary to go to the expense of putting in condensation gutters.

This building is described in the Railway and Engineering Review, March 9, 1901.

For additional details of saw tooth roofs see Fig. 97.

The cross-section of a locomotive shop for the Eastern Railway of France is shown in Fig. 85. The entire building is made of fireproof materials, the framework is of iron and the roof of sheet metal and glass. The building extends from east to west and has a saw tooth roof, with the shorter leg facing north, and glazed with crinkled glass. The floor is made of treated oak cubes measuring 3.94 in. on the edge, set with the grain vertical, on a bed of river sand about 8 in. thick. The saw tooth roof is well suited to structures of this class.

Example of Ketchum's Modified Saw Tooth Roof.—The modified form of saw tooth roof described above was proposed by the author in the first edition (1903). This form of saw tooth roof has recently (1905) been used in the paint shops of the Plank Road Shops of the Public Service Corporation of New Jersey, Newark, N. J. The building proper is 135 feet wide by 354 feet long. The main trusses are of the modified saw tooth type with 44-ft. spans and a rise of ¼, and are spaced 16 ft. centers. The general details of one of the main trusses are shown in Fig. 84a.

The building has an independent steel framing with brick curtain walls on the exterior. Pilasters 24 in. by 20 in. are placed 16 feet apart under the ends of the trusses, the intermediate curtain walls being 12 inches thick.

The roof is a 5-ply slag roof laid on 2-in. tongued and grooved spruce sheathing, which is spiked to 2 in. × 5 in. spiking strips, which are bolted to 8-in. channel purlins spaced 6 feet centers. The slag roofing is laid to comply with standard specifications as described in Part IV.

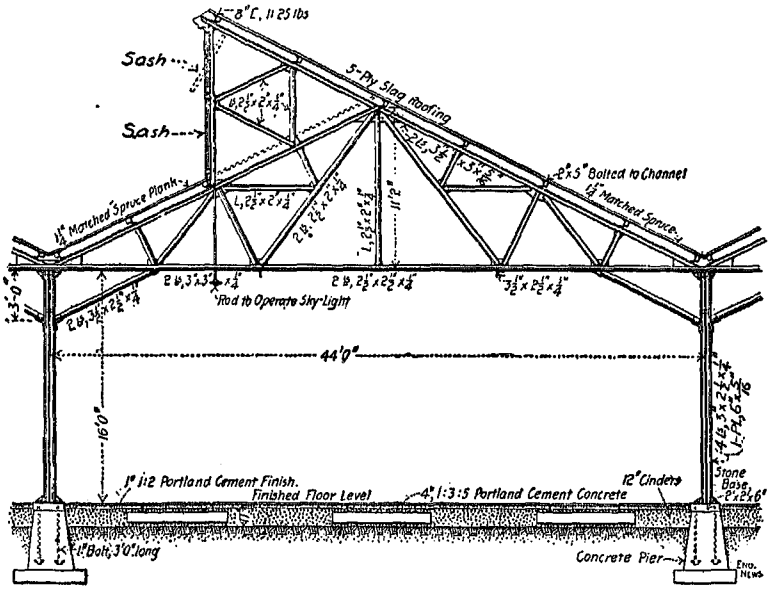


FIG. 84a. MODIFIED SAW TOOTH ROOF, PAINT SHOP, PUBLIC SERVICE CORPORATION.

The roof water is carried down 5-in. cast iron leaders attached to alternate interior columns.

The sash in the vertical leg are in two rows, the upper row being hinged at their centers, thus providing ample ventilation. Condensation gutters are placed below the vertical leg to take the drip. The skylight area is about 20 per cent of the roof area, the window area is about 45 per cent of the outside walls, while about 28 per cent of the entire outside surface of the building is of glass. All glazing is of ⅛-in. ribbed wire glass, with the ribs placed vertical.

The skylight frames and moldings are made of No. 24 galvanized iron, while the entire roof is flashed with 16-oz. copper sheets, 4 feet wide, and counter flashed with sheet lead.

Louisville & Nashville R. R. Shops.—The saw tooth roof shown in Fig. 84b was used in the South Louisville shops of the Louisville & Nashville R. R. The roof covering is composed of composition roofing on the pitched roof and asphalt and gravel roofing on the flat portions laid on 1¾" dressed and matched sheathing.

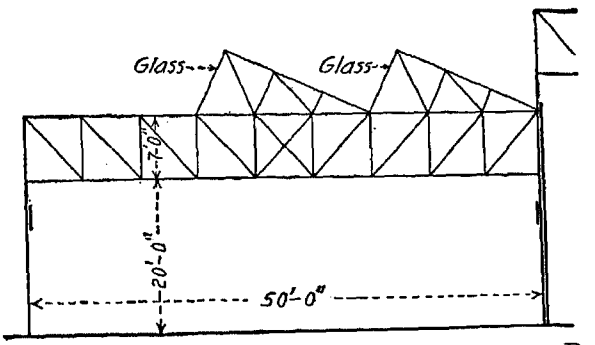


FIG. 84b. SAW TOOTH ROOF, LOUISVILLE & NASHVILLE R. R. SHOPS.

The short leg of the saw tooth is glazed with ribbed wire glass, and the building is ventilated by means of 12-inch circular ventilators placed in the peak of the saw teeth and spaced 30′ 2″.

Saw Tooth Roof of the Ingersoll-Sargent Drill Co.—The design of the saw tooth roof used on the Rock Drill building is shown in Fig. 84c. The floors are of concrete and the roof is of reinforced concrete covered with felt and slag. Ribbed glass is used in the glazed tooth. It will be noticed that electric motors for driving the machinery are

placed on small platforms resting on the lower chords of the roof trusses.

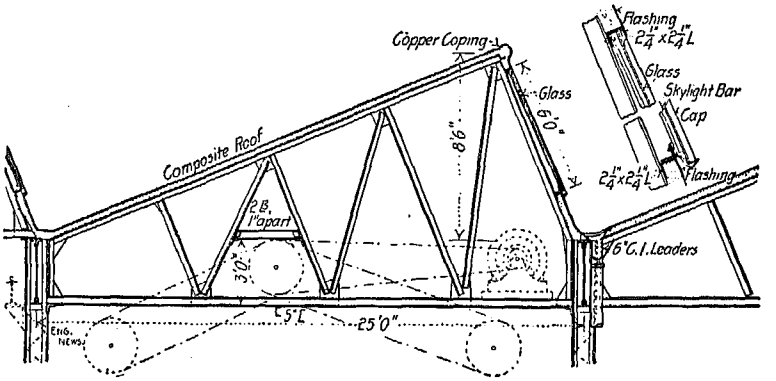


FIG. 84c. SAW TOOTH ROOF FOR ROCK DRILL BUILDING.*

Erecting and Machine Shop of P. & L. E. R. R.—The locomotive-erecting and machine shop of the P. & L. E. R. R., at McKees Rocks is 533′ × 168′ 1″, and is designed with a self-supporting frame with brick curtain walls.

The details of the roof are shown in the cross-section in Fig. 84d, the machine shop being well lighted by three saw tooth windows. The roofing is asphalt and felt laid on 1⅜″ tongued and grooved boards. The gutters are heavily flashed with asphalt and felt. The roof water in the saw tooth part is carried by 4″ conductors to 5″ vertical discharge pipes, attached to alternate columns (40 ft. apart) as shown. The details of the saw tooth windows are given in Fig. 157a. The floors have a 1–3–5 Portland cement concrete base 4″ thick, on this are placed five layers of felt saturated with asphalt for waterproofing. This is covered with a layer of dry sand about 5″ thick and the 4″ × 3⅞″ floor stringers are well bedded in the sand. The wearing floor consists of a sub-floor of 2¾″ yellow pine and a top floor of 1⅛″ tongued and grooved maple. Wire boxes are put in as shown in Fig. 84d to carry power and light wires.

* Engineering News, June 20, 1905.

Partial Elevation of North Side.
End Elevation.
Machine Shop
Machine Shop
Erecting Shop
Cross Section Showing Construction of Floor.

FIG. 84d.

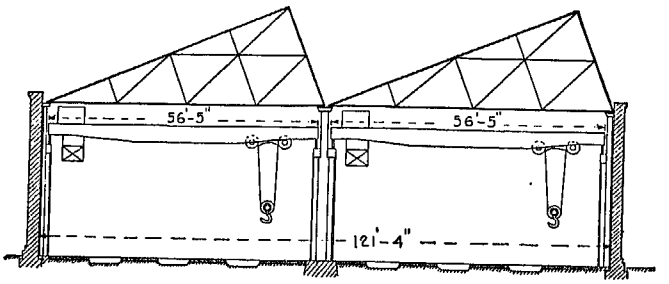


FIG. 85. LOCOMOTIVE SHOP, EASTERN RAILWAY OF FRANCE.

A few of the forms of trusses in common use where ventilation and light are provided for are shown in Fig. 86. The Fink truss with

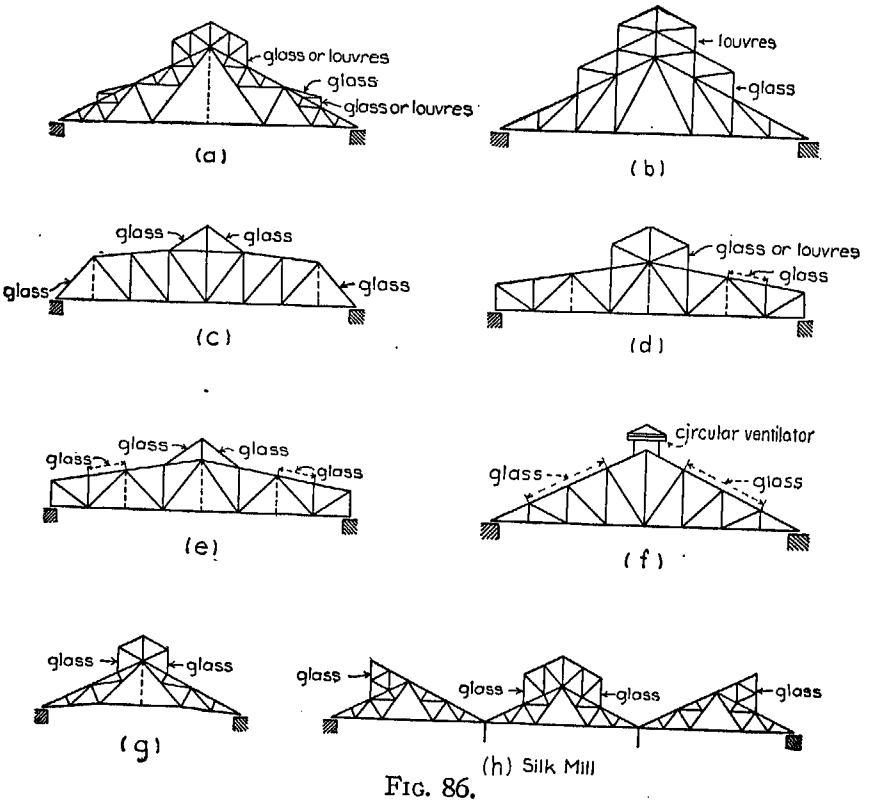


FIG. 86.

monitor ventilator and skylights in the roof shown in (a), is a favorite type for shops; truss (b) with double monitor ventilator is especially adapted to round house construction; trusses (c) and (e) are adapted to shop and factory construction where a large amount of light is desired, ventilation being obtained by means of circular ventilators; truss (d) is similar to (c) and (e), but allows of better ventilation; truss (f) has skylights in the roof and has circular ventilators placed along the ridge of the roof; truss (g) is the type in common use for blacksmith shops, boiler houses, and roofs of small span. The "silk mill" roof shown in (h) was used by the Klots Throwing Co. in their silk mill at Carbondale, Pa. The spans of the three trusses are 48′ 8″ each, with a clerestory of 13′ 9″ in the monitor ventilators, which are glazed with glass 11′ 0″ high. The monitors face east and west, allowing a maximum amount of direct sunlight in the morning and evening, and none at midday. This roof has given very satisfactory results, however, it would seem to the author that it would be necessary to use shades, and that there would be shadows in the building. The trusses in this building are spaced 10′ 6″ apart and support the plank sheathing which carries the roof, no purlins being used. The shafting to run the machinery in this building is placed in a sub-basement; a method much more economical and convenient than the common one of suspending the shafting from the trusses.

Pitch of Roof.—The pitch of a roof is given in terms of the center height divided by the span; for example a 60-ft. span truss with ¼ pitch will have a center height of 15 ft. The minimum pitch allowable in a roof will depend upon the character of the roof covering, and upon the kind of sheathing used. For corrugated steel laid directly on purlins, the pitch should preferably be not less than ¼ (6″ in 12″), and the minimum pitch, unless the joints are cemented, not less than ⅕. Slate and tile should not be used on a less slope than ¼ and preferably not less than ⅓. The lap of the slate and tile should be greater for the less pitch. Gravel should never be used on a roof with a greater pitch than about ⅕, and even then the composition is very liable to run. As-

phalt is inclined to run and should not be used on a roof with a pitch of more than, say, 2 inches to the foot. If the laps are carefully made and cemented a gravel and tar or asphalt roof may be practically flat; a pitch of ¾ to 1 inch to the foot is, however, usually preferred. Tin may be used on a roof of any slope if the joints are properly soldered. Most of the patent composition roofings give better satisfaction if laid on a roof with a pitch of ⅕ to ¼. Shingles should not be used on a roof with a pitch less than ¼, and preferably the pitch should be ⅓ to ½.

Pitch of Truss.—There is very little difference in the weight of Fink trusses with horizontal bottom chords, in which the top chord has a pitch of ⅕, ¼, or ⅓. The difference in weight is quite noticeable, however, when the lower chord is cambered; the truss with the ⅓ pitch being then more economical than either the ⅕ or the ¼ pitch. Cambering the lower chord of a truss more than, say, 1-40 of the span adds considerable to the weight. For example the computed weights of a 60-ft. Fink truss with a horizontal lower chord, and a 60-ft. Fink truss with a camber of 3 feet in the lower chord, showed that the cambered truss weighed 40 per cent more for the ¼ pitch and 15 per cent more for the ⅓ pitch, than the truss having the same pitch with horizontal lower chord. It is, however, desirable for appearance sake to put a slight camber in the bottom chords of roof trusses, for the reason that to the eye a horizontal lower chord will appear to sag if viewed from one side.

In deciding on the proper pitch, it should be noted that while the ⅓ pitch gives a better slope and has a less snow load than a roof with ¼ or ⅕ pitch, it has a greater wind load and more roof surface. Taking all things into consideration ¼ pitch is probably the most economical pitch for a roof. A roof with ⅓ pitch is, however, very nearly as economical, and should preferably be used where corrugated steel roofing is used without sheathing, and where the snow load is large.

Economic Spacing of Trusses.—The weight of the trusses and columns per square foot of area decreases as the spacing increases, while

the weight of the purlins and girts per square foot of area increases as the spacing increases. The economic spacing of the trusses is a function of the weight per square foot of floor area of the truss, the purlins, the side girts and the columns, and also of the relative cost of each kind of material. For any given conditions the spacing which makes the sum of these quantities a minimum will be the economic spacing. It is desirable to use simple rolled sections for purlins and girts, and under these conditions the economic spacing will usually be between 16 and 25 feet. The smaller value being about right for spans up to, say, 60 feet, designed for moderate loads, while the greater value is about right for long spans, designed for heavy loads.

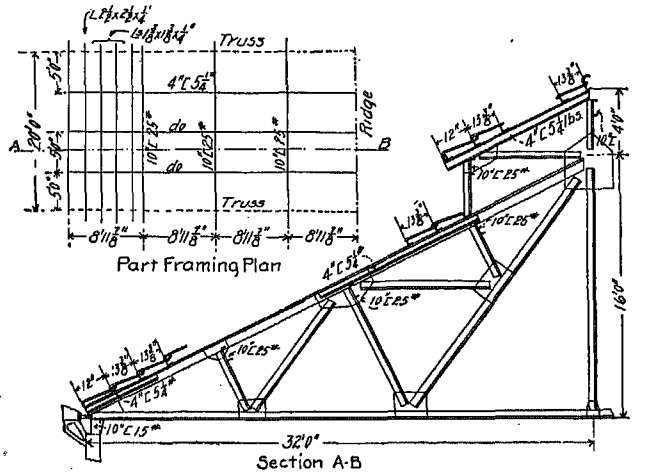


FIG. 87. STEEL ROOF COVERED WITH LUDOWICI TILE.

Calculations of a series of simple Fink trusses resting on walls and having a uniform span of 60 feet, and different spacings gave the least weight per square foot of horizontal projection of the roof for a spacing of 18 feet, and the least weight of trusses and purlins combined for a spacing of 10 feet. The weight of trusses per square foot was, however, more for the 10-ft. spacing than for the 18-ft. spacing, so that the actual cost of the steel in the roof was a minimum for a

spacing of about 16 feet; the shop cost of the trusses per pound being several times that of the purlins. Local conditions and requirements

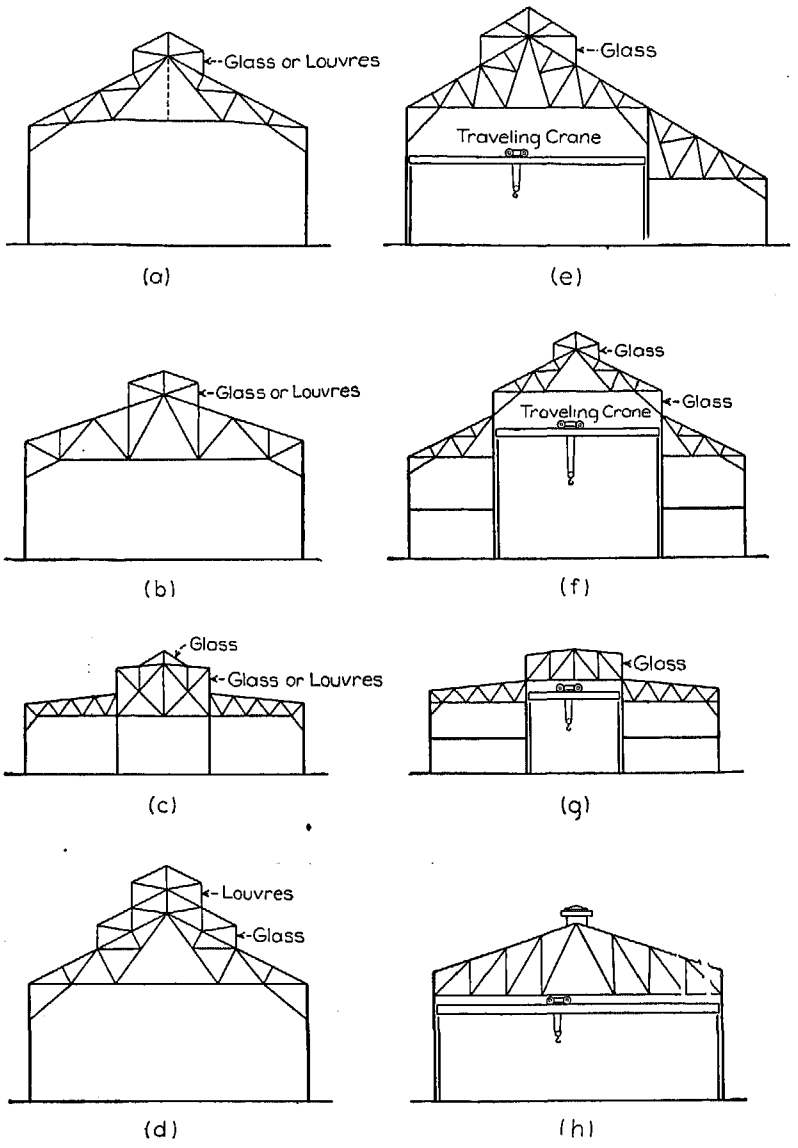
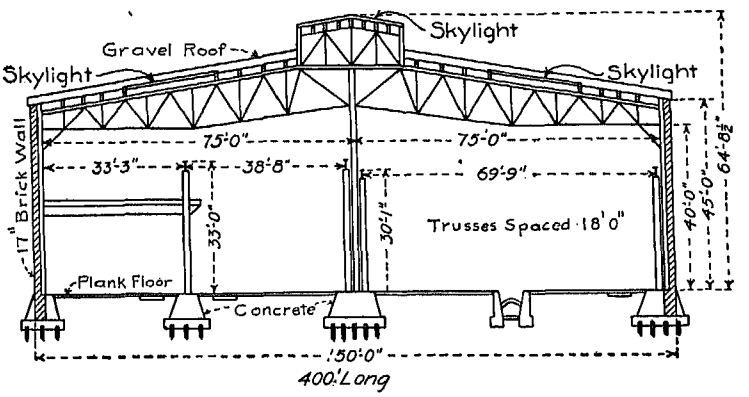


FIG. 88. TYPES OF TRANSVERSE BENTS.

usually control the spacing of the trusses so that it is not necessary that we know the economic spacing very definitely.

For long spans the economic spacing can be increased by using rafters supported on heavy purlins, placed at greater distances than would be required if the roof were carried directly by the purlins. This method is frequently used in the design of train sheds and roofs of buildings where plank sheathing is used to support slate or tile coverings, or where the tiles are supported by angle sub-purlins spaced close together as shown in Fig. 87.

TRANSVERSE BENTS.—The proper cross-section for a mill building will depend upon the use to which the finished structure is to



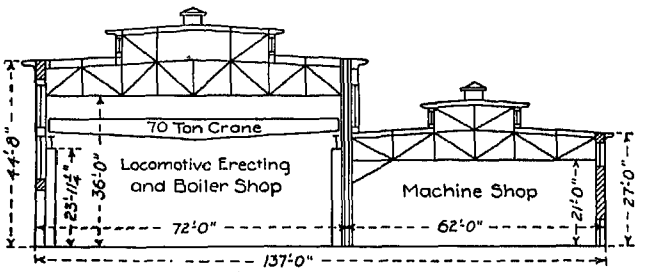
Locomotive Shop-Oregon Short Line

FIG. 89.

be put. A number of the common types of transverse bents are shown in Fig. 88. Transverse bents (a), (b), (d) and (h) are commonly used for boiler houses, shops and small train sheds. Where a traveling crane is desired, the crane girders are commonly suspended from the trusses in the bents referred to, although the crane may be made to span the entire building as in (h). Transverse bent (d) was used for a round house with excellent results. Transverse bents (f) and (g) are quite commonly used where it is desired that the main part of the building be open and be provided with a traveling crane that will sweep

the building, while the side rooms are used for lighter tools and miscellaneous work. Transverse bent (c) may be used in the same way as (g), by supplying a traveling crane. Transverse bent (e) is very often used for shops.

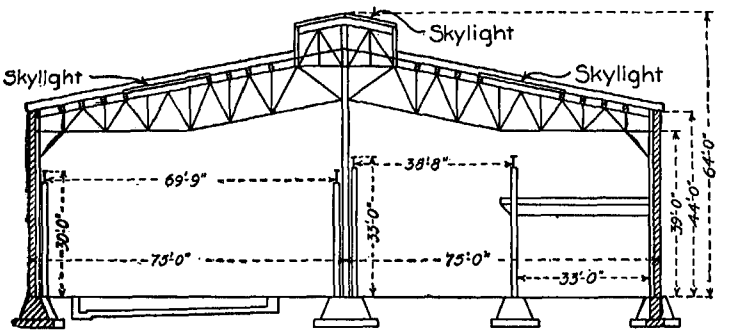
Cross-sections of the locomotive shops of several of the leading railways are shown in Figs. 89 to 92, inclusive, and the locomotive shops of the A. T. & S. F., and the Philadelphia and Reading Railroads are described in detail in Part IV. For the most part these buildings



Locomotive Shop - St.L.,I.M.&S.

FIG. 90.

are built with self-supporting frames, and have brick walls built outside the framing. The arrangement of the cranes, provisions for light-



Locomotive Shop - Union Pacific.

FIG. 91.

ing and ventilating, and the main dimensions are shown in the cuts and need no explanation.

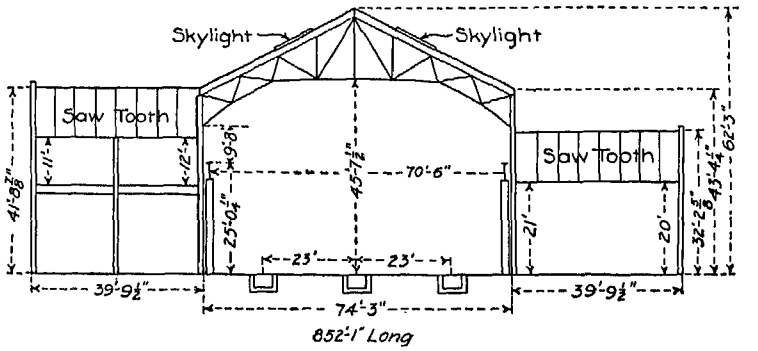
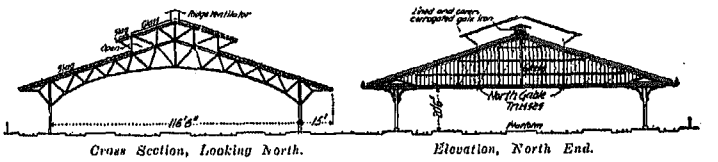


FIG. 92.

A cross-section and end view of the train shed of the Richmond Union Passenger Station are shown in Fig. 93. Riveted trusses are



Train Shed—Richmond Union Passenger Station.

FIG. 93.

quite generally used in train sheds; a notable exception to this statement, however, being the trusses for the new train shed of the C. R. I. & P., and L. S. & M. S. Railways in Chicago. The trusses in this structure have a length of span of 207 ft., a rise of the bottom chord of 40 ft. and a depth of truss at the center of 25 ft. The trusses are pin connected, the compression members being built up channels and the tension members eye-bars. The building is described in detail in Engineering News, August 6, 1903.

Truss Details.—Riveted trusses are commonly used for mill buildings and similar structures. For ordinary loads, the upper and lower

chords, and the main struts and ties are commonly made of two angles placed back to back, forming a T-section, the connections being made by means of plates. The upper chord should preferably be made of unequal legged angles with the short legs turned out. Sub-struts and ties are usually made of one angle. Flats should not be used. Where a truss member is made of two angles placed back to back, the angles should always be riveted together at intervals of 2 to 4 feet.

Trusses that carry heavy loads or that support a traveling crane or hoist, are very often made with a lower chord composed of two channels placed back to back and laced or battened, and are sometimes made with channel chord sections throughout (see Fig. 175).

When the purlins are not placed at the panel points of the truss the upper chord must be designed for flexure as well as for direct stress. The section in most common use for the upper chord, where the purlins are not placed at the panel points, is one composed of two angles and a plate as shown in (c) Fig. 96.

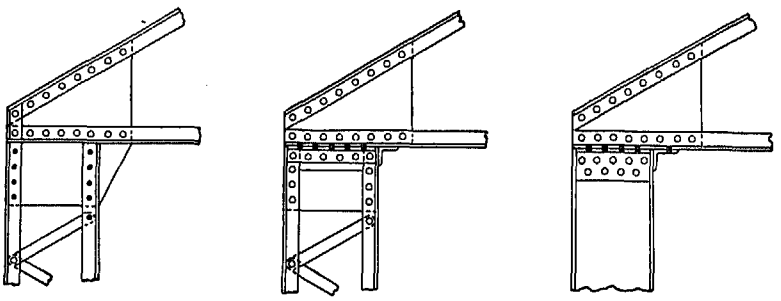


FIG. 94.

Trusses may be fastened to the columns by means of a plate as shown in (a) Fig. 94, or by means of connection angles as shown in (b) and (c). The first method is to be preferred on account of the rigidity of the connection, and the ease with which the field connection can be made.

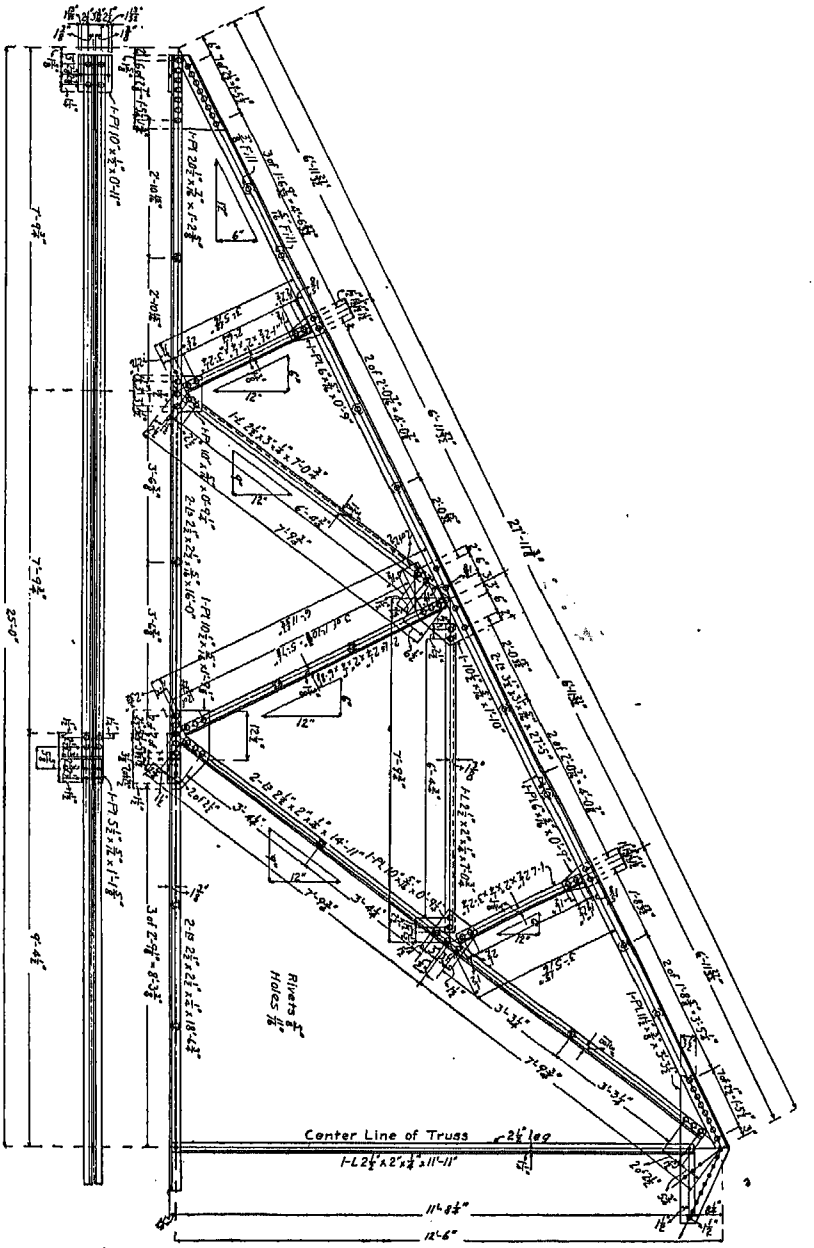


FIG. 95. SHOP DRAWINGS FOR A STEEL ROOF TRUSS.

Trusses supported directly on masonry walls have one end supported on sliding plates for spans up to about 70 feet; for greater lengths of span one end should be placed on rollers, or should be hung on a rocker. Trusses for mill buildings should be made with riveted rather than with pin connections, on account of the greater rigidity of the riveted structure. The complete shop drawings of a truss for the machine shop at the University of Illinois, are shown in Fig. 95. This truss is more completely detailed than is customary in most bridge shops. The practice in many shops is to sketch the truss, giving main dimensions, number of rivets and lengths of members, depending on the

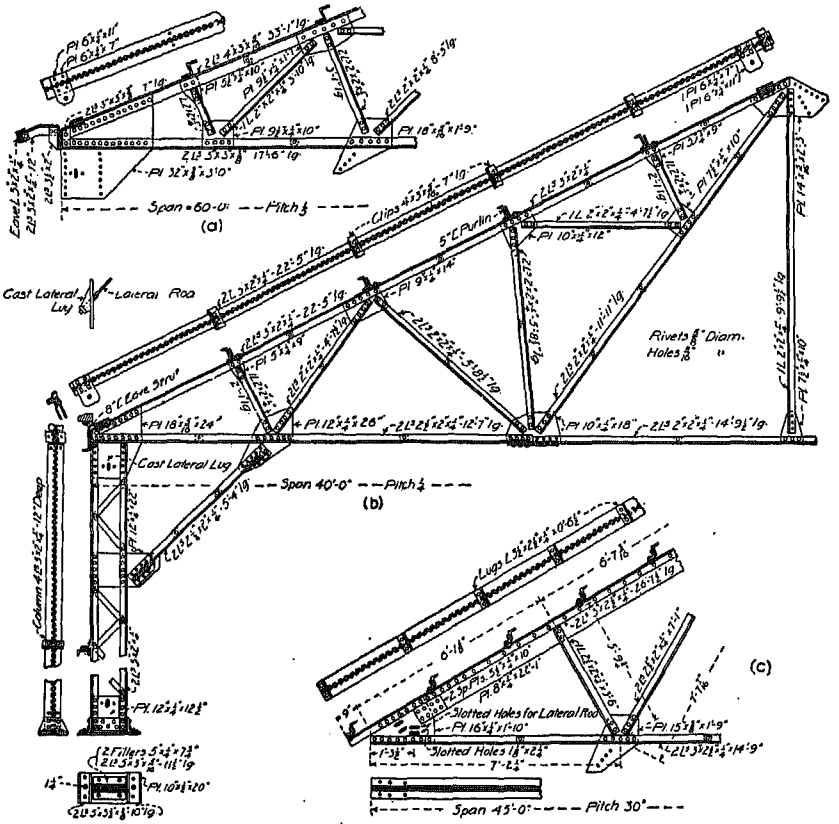


FIG. 96.

templet maker for the rest. In Fig. 95 the rivet gage lines are taken as the center lines. This is the most common practice, although many use one leg of the angle as the center line in secondary members. The latter method has the advantage of reducing the length of connection plates without introducing secondary stresses that are liable to be troublesome.

The detail drawings of a transverse bent are shown in Fig. 96. The common methods of attaching purlins and girts, and of making lateral connections are also shown. The fan type of Fink Truss shown in Fig. 96 is quite commonly used where an odd number of panels is desired, and makes a very satisfactory design. The details of the end connection of a 60-ft. span truss are shown in (a), and of a 45-ft. span truss with a reinforced top chord are shown in (c), Fig. 96. The method of reinforcing the top chord shown (c) is the one most commonly employed where purlins are not placed at the panel points. The method of making lateral connections for the lateral rods shown in (c) is not good, for the reason that it brings bending stresses in a plate which is already badly cut up.

The detail drawings of a saw tooth roof bent for the Mathiessen & Hegeler Zinc Works, LaSalle, Ill., are shown in Fig. 97. This building was erected in 1899 along the lines suggested by an experience with a similar saw tooth roof building erected in 1874. The building was designed by Mr. August Ziesing, Vice President American Bridge Co., and was erected by the American Bridge Co.

The following description is from a personal letter from Mr. Julius Hegeler of the firm of Mathiessen & Hegeler, to the author in reply to a request for plans: "The cast iron gutters are fastened to the purlins and roof boards by spikes through holes in the gutters (holes are not shown in the drawing); on account of their slope, however, hardly any fastening is necessary. These gutters are so placed that the galvanized iron down spouts are next to the posts, there being two down spouts at each post. The condensation gutters are fastened to the gutters and empty into the down spouts. Ice has never caused any trouble by forming in the gutters."

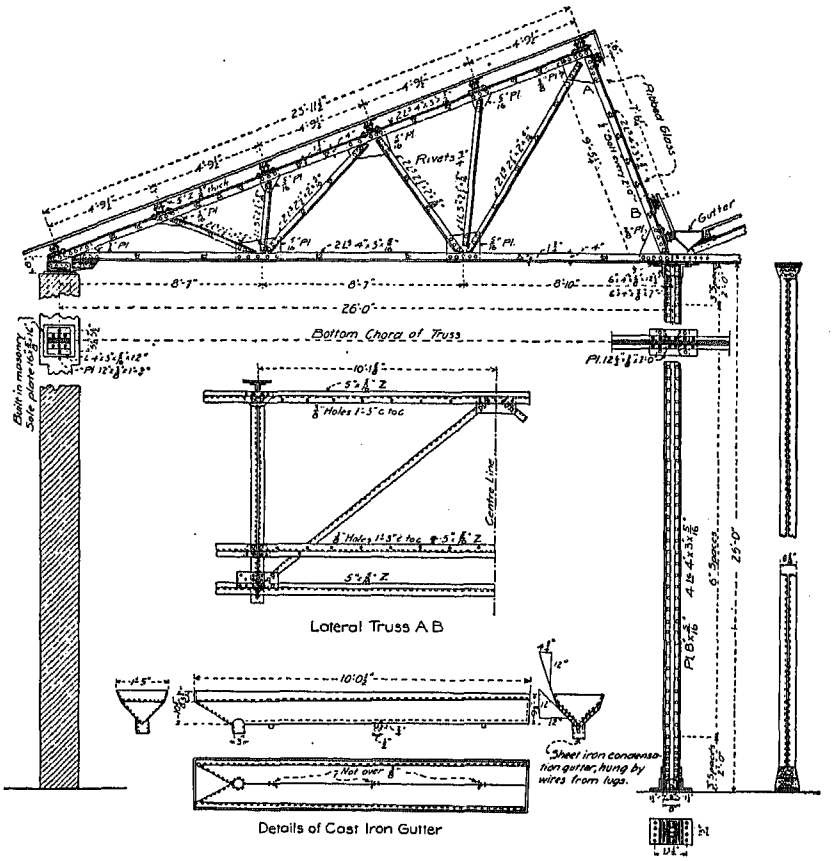


FIG. 97. CROSS-SECTION OF THE SHOPS OF THE MATHIESSEN & HEGELER ZINC WORKS, LASALLE, ILL.

The original saw tooth roof shop built by this firm in 1874 is still in use, and is one of the first, if not the first, saw tooth roofs built in America.

COLUMNS.—The common forms of columns used in mill buildings are shown in Fig. 98. For side columns where the loads are not excessive, column (g) composed of four angles laced is probably the best. In this column a large radius of gyration about an axis at right angles to the direction of the wind is obtained with a small amount

of metal. The lacing should be designed to take the shear, and should be replaced by a plate, (f) Fig. 98, where the shear is excessive, or where the bending moment developed at the base of the column requires the use of excessive flanges. The I beam column (h) makes a good side column where proper connections are made, and is commonly used for end columns (see Fig. 81). The best corner column is made of an equal legged angle with 4, 5 or 6-in. legs, (i) Fig. 98. Details for the bases of the three columns above described are shown in Fig. 99.

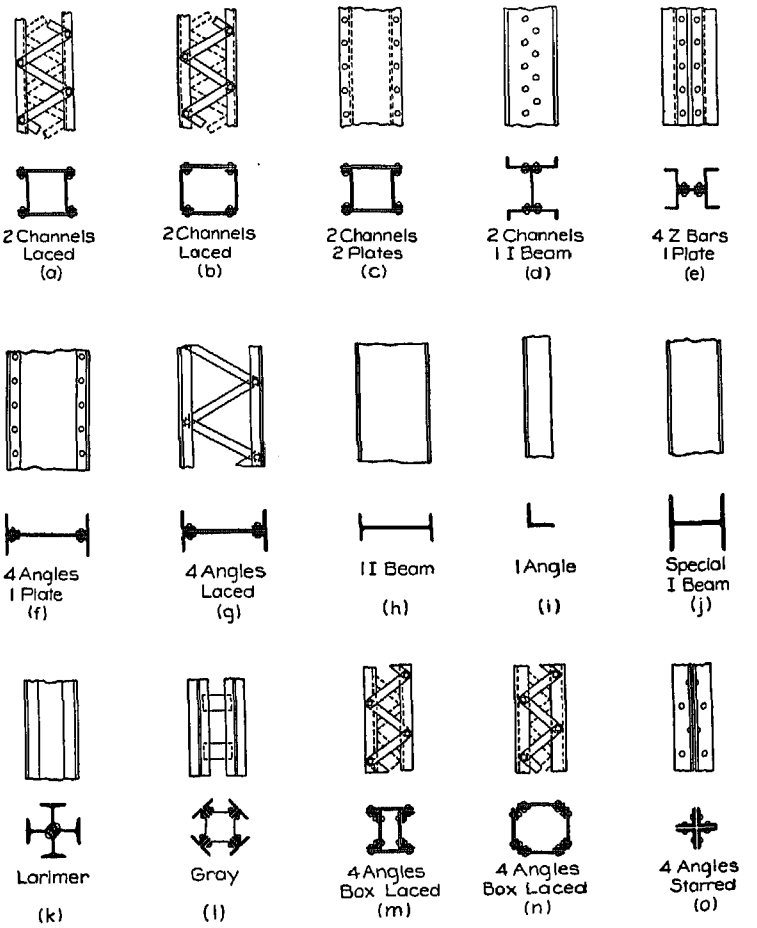


FIG. 98. TYPES OF MILL BUILDING COLUMNS.

Columns made of two channels laced, or two channels and two plates, are used where moderately heavy loads are to be carried. Channel column (a), with channels turned back to back and laced, is the form most commonly used; column (b), with the backs of the channels turned out and laced, gives a better chance to make connections and can be made to enter an opening without chipping the legs of the channel; column (c) is a closed section and is seldom used on that account. The cost of the shop work on column (b) was formerly considerably more than for column (a), for the reason that it was impossible to use a power riveter for driving all the rivets. A pneumatic riveter is now made, however, that will drive all the rivets in column (b), and the shop cost for columns (a) and (b) are practically the same.

Where very heavy loads are to be carried, columns (d) or (e) are often used. Column (d), composed of two channels and one I beam, is a very economical column and is quite often used as a substitute for the Z-bar column shown in (e), for the reason that it can be built up out of the material that is in stock or that can be easily obtained. Connections for beams are easily and effectively made with either columns (d) or (e). The special I beam column (j), with flanges equal to the depth of the beam, is now being rolled in Germany by the use of a process patented by an American, Mr. Henry Grey. This column makes an almost ideal column for heavy loads, since it has all the advantages of the Z-bar column with a very much smaller shop cost. The Larimer column (k) is a patented column manufactured by Jones & Laughlins, and is used by their patrons quite extensively. The Gray column (l) is a patented column and is but little used. Columns made of four angles box-laced, are used where extremely light loads are carried by very long columns. The shop cost of column (m) is somewhat less than that of column (n), although with small angles there is no difficulty in riveting (n) with a machine riveter. Column (o) is a very poorly designed column, for the reason that the radius of gyration is very small for the area of a cross-section of the column. Columns made of two angles "starred" and fastened at intervals of two or three

feet by means of batten plates, are quite frequently used for light loads.

Column Details.—The details of a 4-angle laced column attached to a truss are shown in Fig. 96; and the details of a 4-angle plate column are shown in Fig. 97. The details of bases for 4-angle, I beam and angle columns are shown in Fig. 99.

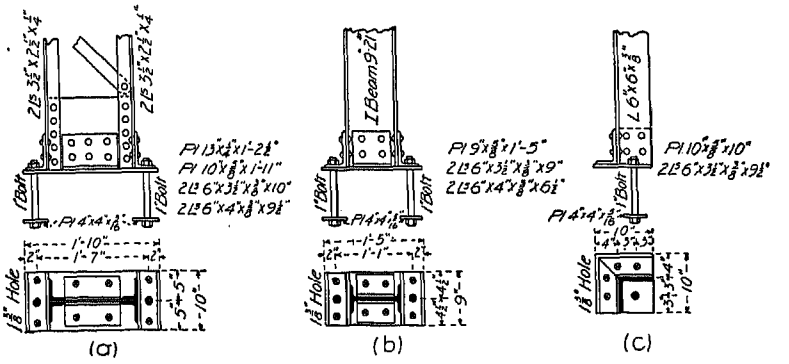


FIG. 99.

Shop details of a 4-angle column are shown in Fig. 100. This column was designed for a mill building with a span of 60 feet, trusses spaced 16 feet apart. The long legs of the angles are placed out, to give a larger radius of gyration about an axis at right angles to the direction of the wind. The details of a 4-angle and plate column, designed to carry a crane girder as well as the roof, are shown in Fig. 101.

The details of a heavy column composed of two channels placed back to back and laced, are shown in Fig. 102; the lacing is heavy and is well riveted. The bent plate connections for the anchor bolts on this column are very satisfactory. This is one of the columns used in the A. T. & S. F. R. R. shops at Topeka, Kas., to carry the crane girders.

The shop details of a light channel column are shown in Fig. 103. The single lacing alternates on the two sides of the column. The various details of the columns can be seen, and require no explanation.

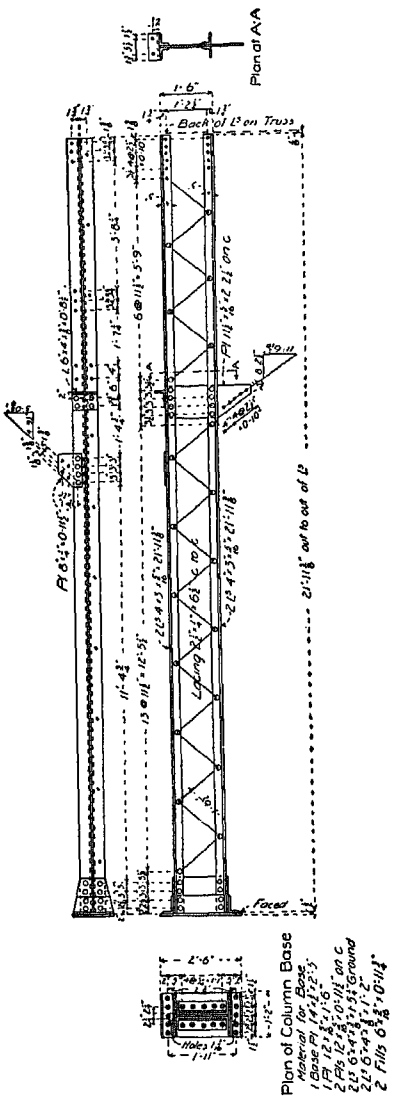


FIG. 100.

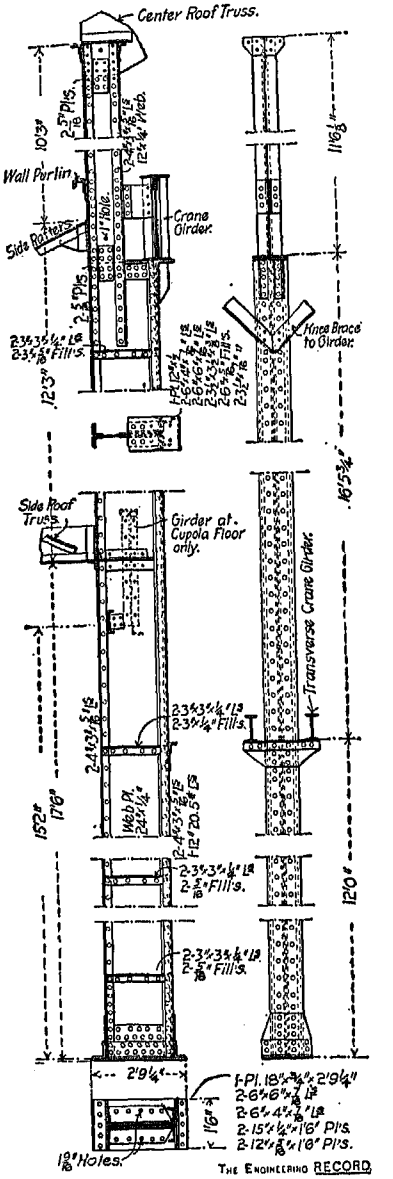


FIG. 101.

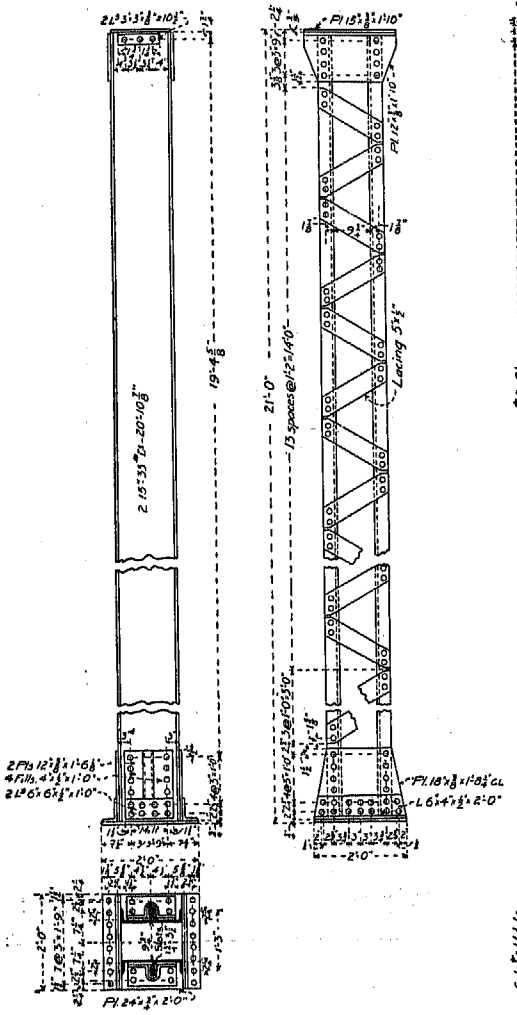


FIG. 102.

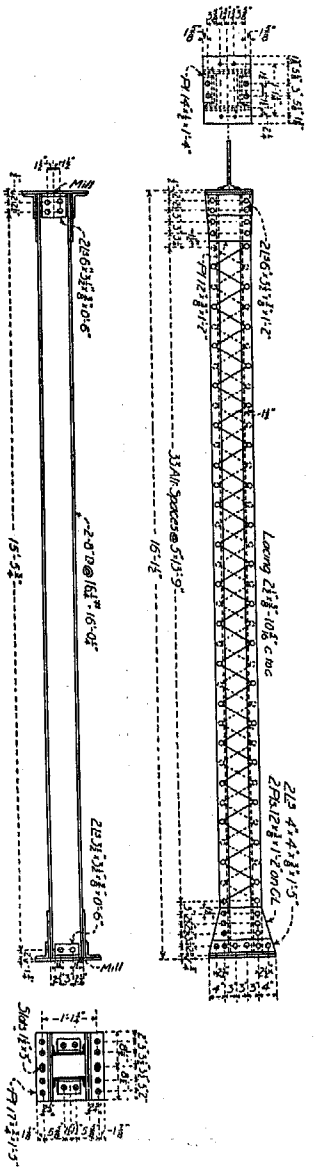


FIG. 103.

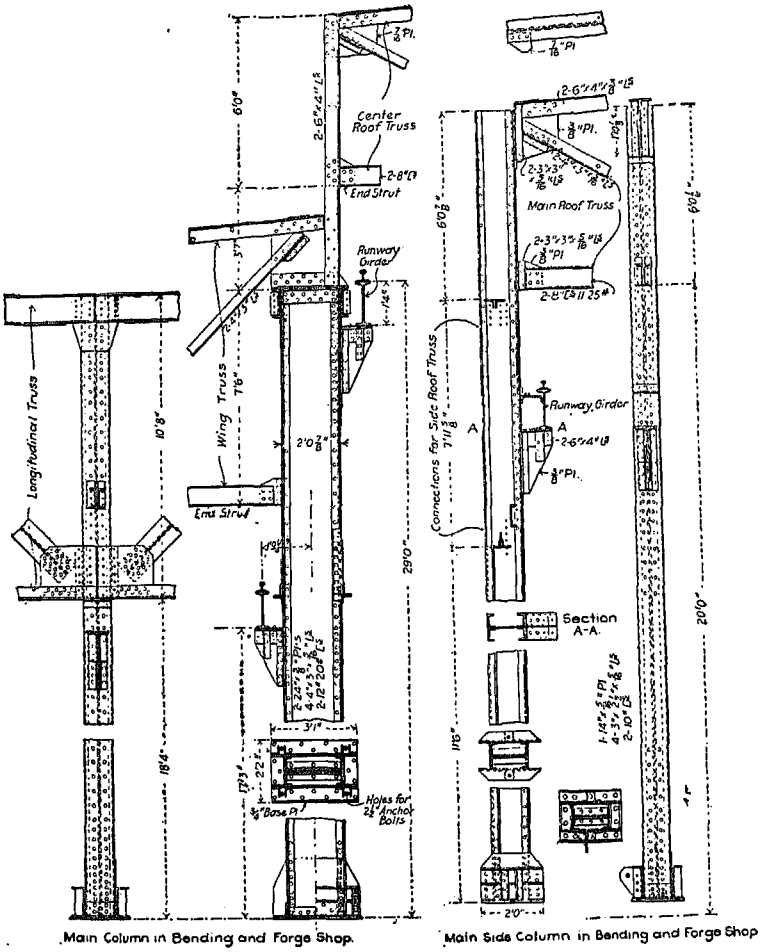


FIG. 103a. FIG. 103b.

Details of columns designed to carry crane girders are shown in Figs. 103a, 103b, 103c, and 103d, which have been taken from the Engineering Record.

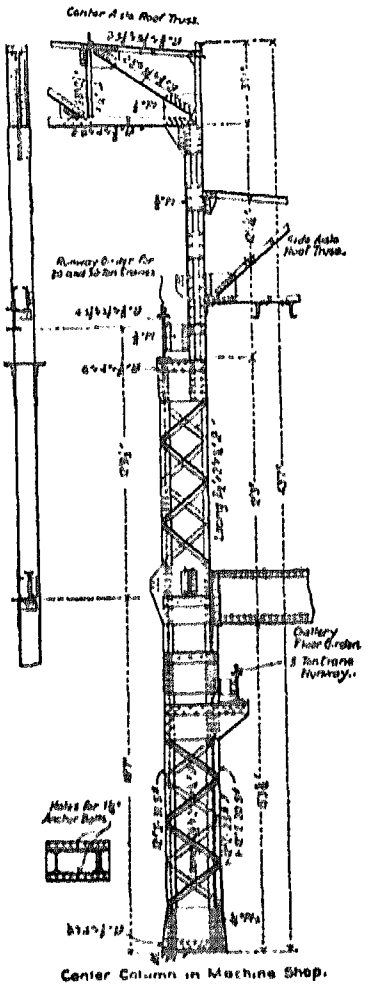


FIG. 103c.

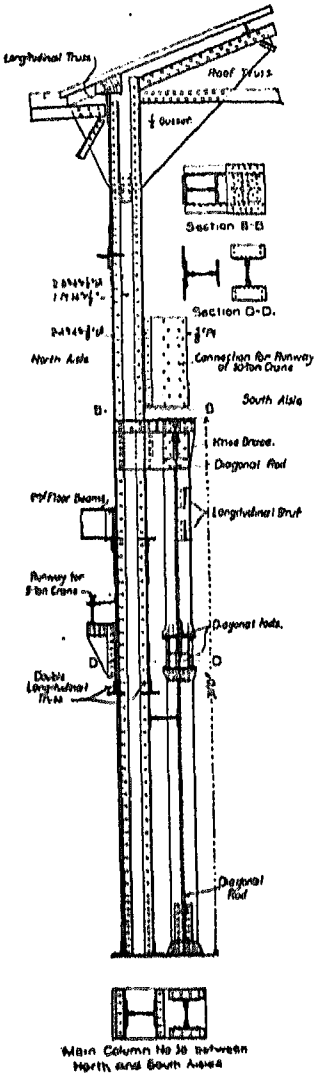
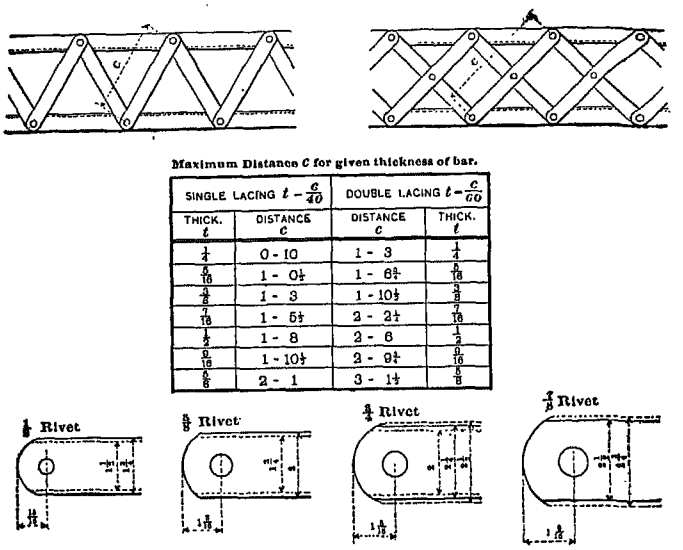


FIG. 103d.

The American Bridge Company's specifications for lattice bars for single and double lacing are shown in Fig. 104.



Maximum Distance C for given thickness of bar.

SINGLE LACING $t = \frac{c}{40}$		DOUBLE LACING $t = \frac{c}{60}$	
THICK. t	DISTANCE c	DISTANCE c	THICK. t
$\frac{1}{4}$	0 - 10	1 - 3	$\frac{1}{4}$
$\frac{5}{16}$	1 - 0½	1 - 6¾	$\frac{5}{16}$
$\frac{3}{8}$	1 - 3	1 - 10½	$\frac{3}{8}$
$\frac{7}{16}$	1 - 5½	2 - 2¼	$\frac{7}{16}$
$\frac{1}{2}$	1 - 8	2 - 6	$\frac{1}{2}$
$\frac{9}{16}$	1 - 10½	2 - 9¾	$\frac{9}{16}$
$\frac{5}{8}$	2 - 1	3 - 1½	$\frac{5}{8}$

FIG. 104.

Single lacing should make an angle of not less than 60 degrees, and double lacing, riveted at the center, not less than 45 degrees with the axis of the member. These specifications are standard.

The properties of angles, I beams and channels, and of Z-bar, Larimer and Gray columns are given in the manufacturers handbooks. The moment of inertia of two channels placed back to back and laced, as in (a) or (b) Fig. 98, about an axis parallel to the webs and through the center of gravity of the section, is given by the formula

$$I = 2\,I' + 2\,Ad^2$$

where I' = moment of inertia of one channel about an axis through its center of gravity and parallel to the given axis, A = the area of one channel, and d = distance from the center of gravity of one channel to the center of gravity of the column. The lacing is omitted in finding the moment of inertia and the area of the section. The moment of inertia of the column about an axis perpendicular to the webs is equal

to twice the moment of inertia of one channel, which may be found in the table of properties of channels given in the handbooks.

Having the moment of inertia I, the radius of gyration of the column is given by the formula

$$r = \sqrt{\frac{I}{A}}$$

With channels placed back to back and laced, the radii of gyration about the two axes are equal when the clear distance is equal to about 3 inches for 5-in. channels, and 10 inches for 15-in. channels. A common rule is to space the channels about eight-tenths the depth. With channels placed with backs out and laced, the radii of gyration about the two axes are equal when the clear distance is about equal to 5 inches for 5-in. channels, and 13 inches for 15-in. channels (see Cambria Steel, 1903 Edition, p. 217).

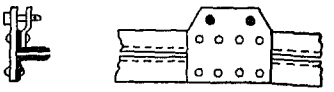
The moment of inertia of a 4-angle laced column, about an axis perpendicular to the lacing and through the center of the post, is given by the formula

$$I = 4I' + 4Ad^2$$

where I' = moment of inertia of one angle about an axis through its center of gravity and parallel to the given axis, A = the area of one angle and d = the distance from the center of gravity of the separate angles to the center of gravity of the column. The moment of inertia about the other axis is found in a similar manner.

STRUTS AND BRACING.—Eave struts are very commonly made of four angles laced, made in the same way as the 4-angle posts, Fig. 100. Eave struts made of single channels are more economical, and are equally as good as the laced struts for most cases. End rafters are commonly made of channels. The sides, ends, upper and lower chords are commonly braced as shown in Fig. 81. The bracing in the plane of the lower chords should preferably be made of members capable of taking compression as well as tension. The diagonal bracing in the plane of sides, ends, and upper chords is commonly composed of rods. Initial tension should always be thrown into diagonal rods by

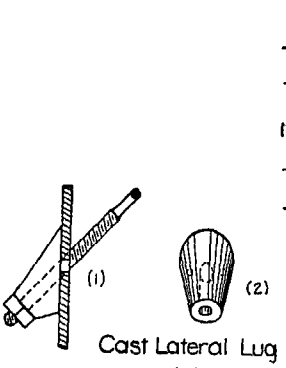
screwing up the turnbuckles or adjustable ends. Stiff bracing should be made short, and should be brought into position for riveting by using drift pins; to accomplish this there should be not less than three rivet holes in each lateral connection. A connection for lateral rods to the chords of trusses is shown in Fig. 105.



Lateral Connection

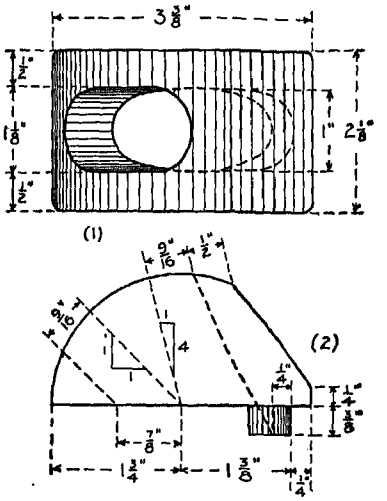
FIG. 105.

Cast lateral lugs for connecting lateral rods to the webs of I beams and to heavy plates are shown in (a) and (b), Fig. 106.



Cast Lateral Lug

(a)



Cast Lateral Lug

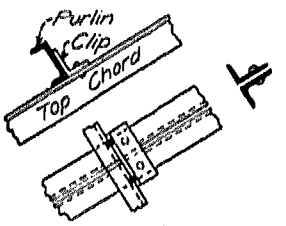
(b)

FIG. 106.

Where rod bracing in the ends and sides of buildings interferes with windows and doors, or where the building is to be left open, portal bracing is used. In the latter case the bents are usually braced in pairs,

although the portal bracing is sometimes made continuous. Stiff bracing is often placed between the trusses in the plane of the center of the building and materially stiffens the structure (see Fig. 175).

PURLINS AND GIRTS.—Purlins are made of channels, angles, Z-bars and I beams, Fig. 111, where simple shapes are used. Channel and angle purlins should be fastened by means of angle lugs as shown in Fig. 107. I beam purlins are very often fastened as shown in the A. T. & S. F. R. R. shops, Fig. 175. Z-bar purlins are bolted directly to the upper chords of the trusses. The channel purlin is the most economical, and the I beam purlin is the most rigid. Girts are made of channels, angles, and Z-bars, and are fastened as shown in Fig. 111. Where the distance between trusses is more than 15 or 16 feet the purlins and girts should be kept from sagging by running ⅜ or ½-inch rods through the centers to act as sag rods, the ends of the rods being fastened to the eaves and ridge (see Fig. 81).



Purlin Clip
FIG. 107.

Where the columns and trusses are placed so far apart that the use of simple rolled shapes is no longer economical, purlins and girts are trussed.

DESIGN OF PARTS OF THE STRUCTURE.—The methods of determining the sizes of the various members in a mill building will be illustrated by a few examples. For a more detailed treatment of this subject, see "Modern Framed Structures" by Johnson, Bryan and Turneaure; "Roofs and Bridges" by Merriman and Jacoby; and other standard works on bridge design.

Manufacturers of structural material issue handbooks which con-

tain tables that give weights, areas of sections, positions of centers of gravity, moments of inertia, radii of gyration, etc., for the shapes manufactured by the different companies. Tables are also given for the resisting moments on pins, the shearing and bearing values of rivets, standard bolts, eye-bars, bridge pins, standard connection angles, bearing plates, minimum size of rivets, spacing of rivets, and many other useful tables. The handbooks best known are as follows, the popular name being given in brackets: Cambria Steel (Cambria), issued by the Cambria Steel Company, Johnstown, Pa.; Pocket Companion (Carnegie), issued by the Carnegie Steel Company, Pittsburg, Pa.; Standard Steel Construction (Jones & Laughlins), issued by Jones & Laughlins, Limited, Pittsburg, Pa.; Steel in Construction (Pencoyd), issued by A. and P. Roberts Company, Philadelphia; and Structural Steel and Iron (Passaic), issued by the Passaic Rolling Mill Company, Paterson, N. J. These books can be obtained for from 50 cts. to $2.00. The American Bridge Company issued, in 1901, a book entitled Standards for Structural Details, for use at its various plants.

The Carnegie handbook was formerly very generally used in designing offices, but recently the supply has been limited so that the Cambria handbook has taken its place in schools and in many offices, and for this reason references will be made to Cambria in obtaining weights, properties of sections, etc. All references to Cambria will be to the 1903 edition.

Design of Trusses.—The method of determining the proper sizes of the truss members will be illustrated by designing a few of the members of the truss in the transverse bent of the mill building shown in Fig. 53; the stresses in which are given in Table VI. The secondary members will be omitted from the truss in the design, as they were in obtaining the stresses.

The material will be assumed to be medium steel and the allowable stresses as given in Appendix I, will be taken. The allowable stresses are as follows:

Tension ..... 16,000 lbs. per sq. in.

Compression ..... $16{,}000 - 70\,l \div r$ lbs. per sq. in.

where l = the length of the member in inches, and r = radius of gyration of member in inches.

Rivets and Pins, bearing	22,000 lbs. per sq. in.
Rivets and Pins, shear	11,000 " " " "
Pins, bending on extreme fibre	24,000 " " " "
Plate Girder webs, shear on net section	10,000 " " " "

Compression Members.

Piece *x*-2. Maximum stress = + 34,300 lbs.

The upper chord will be made of two angles with unequal legs placed back to back, with the shorter legs turned out, and separated by ⅜-inch gusset or connection plates.

Try two 4" × 3" × 5-16" angles. From table on page 187 Cambria, the least radius of gyration, r, is 1.27 inches. The unsupported length of the member is 8.5 feet, and $l \div r = 102 \div 1.27 = 80$. The allowable stress per square inch = $16{,}000 - 70\, l \div r = 16{,}000 - 5{,}600 =$ 10,400 lbs. The area required will be 34,300 ÷ 10,400 = 3.3 sq. in. The combined area of the two angles is 4.18 sq. ins. (page 170 Cambria), which is somewhat large.

Try two 3½" × 3" × 5-16" angles. From the table on page 186 Cambria, r = 1.10 inches; then $l \div r = 93$, and allowable stress is 16,000 − 70 × 93 = 9,490 lbs. per sq. in. Required area = 3.62 sq. in. The area of the two angles is 3.88 sq. in., so the section is sufficient.

To make the two angles act together as one piece it is necessary to rivet them together at intervals, such that the two angles acting singly will be stronger than the two angles acting together. On page 170 Cambria, the least radius of gyration of a 3½" × 3" × 5-16" angle about a diagonal axis is 0.63 inches. The angles must therefore be riveted at least every 0.63 × 93 = 58.6 inches. It is the common practice to rivet angles in compression about every 2½ to 3 feet.

The truss will be shipped in two parts and in order to avoid a splice, and because the difference in the stresses is small, the entire top chord will be made of two 3½" × 3" × 5-16" angles.

Tension Members.

Member 1-2. Maximum stress = — 24,900 lbs.

The net area required is 24,900 ÷ 16,000 = 1.56 sq. in. The gross area of the section must be such, that there will be a net area of not less than 1.56 sq. in. after the area of the rivet holes in any section has been deducted.

Try two 3″ × 3″ × ¼″ angles. It will be necessary to deduct the area of one rivet hole from each angle. The diameter of the rivet hole deducted is taken ⅛ inch larger than the diameter of the rivet before driving. Assuming the rivets as ⅝ inch, it will be necessary to deduct 0.19 sq. in. from each angle (page 310 Cambria). The net area of two 3″ × 3″ × ¼″ angles is 2.88 — 0.38 = 2.50 sq. in. The section is somewhat large, but will be used, for the reason that angles much smaller than these will be deficient in rigidity.

The angles will be riveted together about every 3 feet to make them act as one member.

Member 5-6. Maximum stress = — 5,000 lbs.

The net area required is 5,000 ÷ 16,000 = 0.32 sq. in. The gross area of the section must be such that there will be a net area of at least 0.32 sq. in. after the area of the rivet holes in any section has been deducted.

Try two 2″ × 2″ × ¼″ angles — the minimum angles that can be used under the specifications. Deducting the area of one rivet the net area is 1.88 — 0.38 = 1.50 sq. in. The section appears to be excessively large and one 2″ × 2″ × ¼″ angle will be tried. Where angles in tension are fastened by one leg the specifications require that (paragraph 35) only one leg shall be counted as effective, or the eccentric stress shall be calculated. The net area of the one 2″ × 2″ × ¼ ″ angle when fastened by one leg, will then be ½ of .94 — 0.19 = 0.28 sq. ins., which is insufficient. One 2½″ × 2½″ × ¼″ angle will have a net area of 0.40 sq. in., which will be sufficient. However, since it is preferable to make tension members of symmetrical sections, the member will be made of two 2″ × 2″ × ¼″ angles.

Alternate Tension and Compression.—Where members are subject alternate tension and compression the specifications require that they designed to take each kind of stress, (paragraph 32).

Member 4-y. Maximum stresses = (— 21,300 and + 2600 lbs).

Try two 3" × 3" × ¼" angles — the same as member 1-2. The area required for tension is 21,300 ÷ 16,000 = 1.34 sq. in.

The net area of two 3" × 3" × ¼" angles is 2.88 — 0.38 = 2.50 in. which is ample for tension.

The least radius of gyration is r = 0.93 inches (page 185 Cambria). Length = 108 inches, and $l \div r$ = 117. The allowable stress r square inch = 16,000 — 70 × 117 = 8,810 lbs. Required area = 30 sq. in. The section appears to be large, but it can not be made ich smaller without exceeding the maximum limit of 125 for $l \div r$. vo 3" × 2½" × ¼" angles will be found by a similar calculation to be fficiently large, and will be used.

Member 3-4. Maximum stresses = (— 10,900 and + 13,600 lbs).

Try two 2" × 2" × ¼" angles. The area required to take the ten- on is 10,000 ÷ 16,000 = 0.69 sq. in. The net area of the two gles is 1.88 — 0.38 = 1.50 sq in., which is ample for tension. The st radius of gyration is r = 0.61 inches (page 185 Cambria). The ngth is 108 inches, and $l \div r$ = 177. This is greater than the max- mum allowed of 125, and a larger section must be used.

Try two 3" × 2" × ¼" angles, with short legs out. In this case ÷ r equals 108 ÷ 0.89 = 120. The allowable stress per sq. in. is 6,000 — 70 × 120 = 7,600 lbs. The required area for compression is 3,000 ÷ 7,600 = 1.79 sq. in. The area of the two angles is 2.38 sq. ., which is ample. The section is sufficiently large to take both ten- on and compression, and will be used.

Design of Columns.—Columns must be designed to take the ress due to direct loading, to eccentric loading, and to wind moment. he method of column design will be illustrated by the design of the eward column in the transverse bent shown in Fig. 56; the stresses for hich are given in Table VI.

Direct stress in A-17 = 14,900 lbs.; and bending moment = 924,000 inch-lbs. A 4-angle laced column will be used.

Try four 4″ × 3″ × 5-16″ angles, long legs out, and a depth of 18″ out to out of angles; ⅜″ lacing and connection plates will be used.

The radius of gyration of two 4″ × 3″ × 5-16″ angles with the long legs out, is found on page 189 Cambria to be 1.93 inches.

The moment of inertia of a section of the post about the shorter axis is

$$\begin{aligned} I &= 4\,I' + 4\,Ad^2 \\ &= 4 \times 1.65 + 4 \times 2.09\ (9.00 - 0.76)^2 \\ &= 574.24 \end{aligned}$$

and the radius of gyration is

$$r = \sqrt{\frac{574.24}{8.36}} = 8.3 \text{ inches.}$$

The maximum fibre stress will occur on the windward side of the post and will be found by substituting in formula (30a) to be

$$f = f_2 + f_1 = \frac{14{,}900}{8.36} + \frac{924{,}000 \times 9}{574.24 - \frac{14{,}900 \times 240^2}{280{,}000{,}000}}$$

$$= 1780 + 14{,}560 = 16{,}340 \text{ lbs. per square inch.}$$

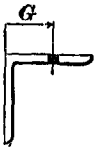
The allowable stress per square inch for direct loads is 16,000 — 70 $l \div r$ = 16,000 — 70 × 29 = 14,000 lbs.; and since the wind moment comes only occasionally we will increase the allowable stress for direct loads by 25 per cent when wind loads are considered, making an allowable stress of 14,000 × 1.25 = 17,500 lbs. per square inch. The section chosen is therefore sufficiently large.

The direct load will have to be carried by the column, and it will be necessary to investigate the column about its longer axis. For this case $l \div r = 240 \div 1.93 = 125$, which is allowable under the specifications, and the section is ample.

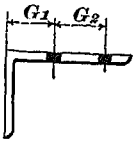
The lacing will be designed to take the shear, which is 5,500 lbs. below and 12,800 lbs. above the foot of the knee brace. The maximum stress in the lacing will be 12,800 × sec 30° = 14,700 lbs. The allowable tensile stress per square inch will be 16,000 × 1.25 = 20,000

TABLE XI.

RIVET SPACING IN ANGLES.



Leg. Inches.	G Inches.	Max. Rivets. Inches.
8	4½	⅞
7	4	"
6	3½	"
5	3	"
4	2¼	"
3½	2	"
3	1¾	"
2½	1⅜	¾
2	1⅛	⅝
1½	⅞	⅜



Leg. Inches.	G_1 Inches.	G_2 Inches.	Max. Rivets. Inches.
8	3	3	⅞
7	2½	3	"
6	2¼	2½	"
5	2	1¾	"
Where 6" Angle Exceeds ¾".			
6	2½	2¼	⅞

TABLE XII.

MAXIMUM SIZE OF RIVETS IN BEAMS, CHANNELS, AND ANGLES.

I-BEAMS.						CHANNELS.			ANGLES.			
Depth of Beam. Inch's.	Weight per Foot. Pounds.	Size of Rivet. Inches.	Depth of Beam. Inches.	Weight per Foot. Pounds.	Size of Rivet. Inches.	Depth of Channel Inches.	Weight per Foot. Pounds.	Size of Rivet. Inches.	Length of Leg. Inches.	Size of Rivet. Inches.	Length of Leg. Inches.	Size of Rivet. Inches.
3	5.5	⅜	15	42.0	¾	3	4.0	⅜	¾	¼	2½	¾
4	7.5	½	15	60.0	¾	4	5.25	½	1	⅜	2¾	⅞
5	9.75	½	15	80.0	⅞	5	6.5	½	1¼	½	3	
6	12.25	⅝	18	55.0	⅞	6	8.0	⅝	$1\frac{5}{16}$	½	3½	1
7	15.0	⅝	20	65.0	1	7	9.75	⅝	1⅜	½	4	1
8	18.0	¾	20	80.0	1	8	11.25	¾	1½	½	4½	1
9	21.0	¾	24	80.0	1	9	13.25	¾	1¾	⅝	5	1
10	25.0	¾				10	15.0	¾	2	⅝	6	1
12	31.5	¾				12	20.50	¾	2¼	¾	7	1
12	40.0	¾				15	33.0	¾	$2\frac{5}{16}$	¾		

TABLE XIII.

RIVET SPACING.

Size of Rivet. Inches.	Minimum Pitch. Inches.	Maximum Pitch at Ends of Compression Members. Inches.	Minimum Pitch in Flanges of Chords and Gird's. Inches.	Distance from Edge of Piece to Center of Rivet Hole.	
				Minimum. Inches.	Usual. Inches.
¼	¾	....	....	....	....
⅜	1⅛	....	....	....	....
½	1½	....	....	....	....
⅝	1⅞	2½	4	15/16	1¼
¾	2¼	3	4	1⅛	1½
⅞	2⅝	3½	4	1 5/16	1¾
1	3	4	4	1½	2

TABLE XIV.

SHEARING AND BEARING VALUE OF RIVETS IN POUNDS.

Diameter of Rivet Inches		Area in Square Inches.	Single Shear at 11000 Lbs.	Bearing Value for Different Thicknesses of Plate in Inches, at 22,000 Pounds per Square Inch.								
Fraction	Decimal			¼	5/16	⅜	7/16	½	9/16	⅝	11/16	¾
⅜	.375	.1104	1210	2060	2580	3090						
½	.500	.1963	2160	2750	3440	4130	4820	5500				
⅝	.625	.3068	3370	3440	4300	5160	6020	6880	7740	8600		
¾	.750	.4418	4860	4130	5160	6190	7220	8250	9280	10320	11340	12380
⅞	.875	.6013	6610	4810	6020	7220	8430	9630	10840	12040	13240	14440
1	1.00	.7854	8640	5500	6880	8250	9630	11000	12380	13750	15130	16500

All bearing values above or to right of upper zigzag lines are greater than double shear. Values below or to left of lower zigzag lines are less than single shear.

lbs. The required net area for tension will be 14,700 ÷ 20,000 = 0.74 square inches. The gross area of a 3″ × ⅜″ bar is 1.125 square inches (page 388 Cambria) and the net area after deducting for one ¾″ rivet is 0.795 square inches (page 310 Cambria) which is sufficient for tension.

The allowable stress for compression is $(16{,}000 - 70\,l \div r)\ 1.25$. The moment of inertia of a 3″ × ⅜″ bar is 0.0135, and the radius of gyration is 0.11 inches. The ends of the lacing bars are practically fixed, and it will be assumed that the length c. to c. of rivets will as a result be shortened by one-half. Then $l \div r = 75$, the allowable stress will be 13,340 lbs. and the required area 1.10 square inches. The lacing bars are therefore sufficient to take either the tension or compression. Lacing bars 2½″ × ⅜″ will be found sufficient below the foot of the knee brace.

The allowable shear on each rivet in the lacing will be (Table XIV) 4860 × 1.25 = 6,075 lbs.; and the allowable bearing will be 6,190 × 1.25 = 7,740 lbs. The stress in the lacing bars below the foot of the knee brace is 5,500 × sec 30° = 6,300 lbs.; the ¾-rivets are all right for bearing but are not quite large enough for shear, however it is so near, that they will be used. Above the foot of the knee brace it will be necessary to increase the thickness of the lacing bars and put two rivets in each connection as shown in Fig. 102, or use a solid plate.

In designing the bases of columns hinged at the base, part of the stresses may be assumed to pass directly to the base plate if the abutting surfaces have been milled; but in columns fixed at the base all of the stresses must be transferred by the rivets. The rivets must be designed to take the direct stress and the stress due to bending moment; the solution is similar to that for anchorage (Fig. 61) and will not be given.

Design of Plate Girders.—The maximum moments and shears are found as described in Chapter X. If the plate girder were designed by means of its moment of inertia, as in the case of rolled sections, about ⅙ of the web would be effective as flange area to take the bending moment; or deducting rivets about ⅛ would be found effective. It is, however, the common practice to assume that all the moment is taken by the flanges, and that all the shear is taken by the web; and this assumption will be made in the discussion which follows.

Flange Stress.—The stress, F, in the flanges at any point in a plate girder is

$$F = M \div h \qquad (80)$$

where M = bending moment in inch-pounds, and h = the distance between centers of gravity of the flange areas (effective depth), (a) Fig. 108.

The net flange area, A, will be $A = F \div f$ where f = the allowable unit stress. The tension flanges of plate girders are designed as above, and the compression flanges are made with the same gross area.

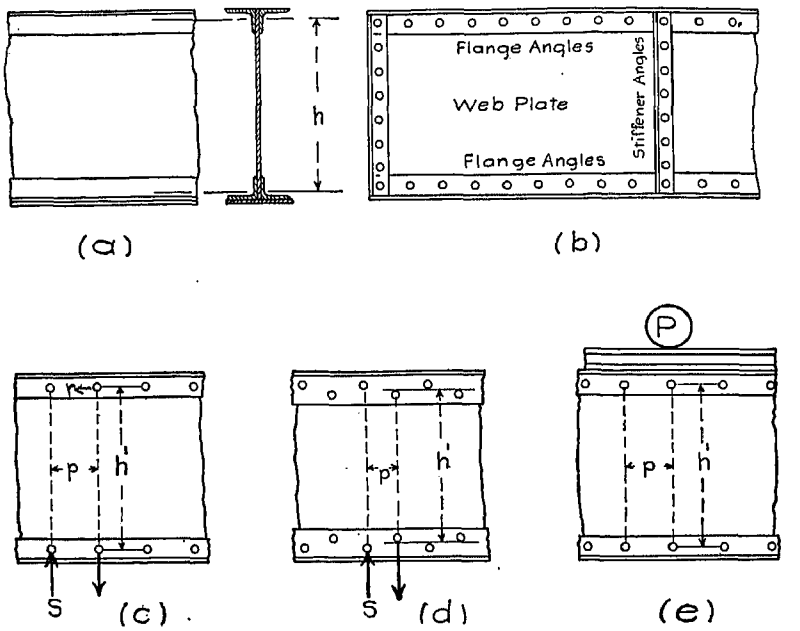


FIG. 108.

Web.—The web plate should not be less than 5-16 of an inch in thickness although ¼-inch plates may be used if provided with sufficient stiffeners. The shear in the web is commonly assumed as uniformly distributed over the entire cross-section of the plate.

Stiffeners.—There is no rational method for the design of stiffeners. If they are placed at distances apart not exceeding the depth of the girder, nor more than 5 feet, where the shearing stress is greater

than given by the formula — allowed shearing stress $= 12{,}500 - 90\,H$, where $H =$ ratio of depth to thickness of web plate, the stiffeners will be near enough together. Where the shearing stress is less than given by the above formula, stiffeners may be omitted or spaced as desired.

Stiffeners are commonly designed as columns, free to move in a direction at right angles to the web, with an allowed stress $P = 12{,}000 - 55\,l \div r$. Stiffeners should be provided at all points of support and under all concentrated loads, and should contain enough rivets to transfer the vertical shear.

Web Splice.—In the plain web splice shown in Fig. 108, the rivets take a uniform shear equal to $S \div n$, where S is total shear, and n is number of rivets on one side of splice, and a shear due to the shearing stress not being applied at the center of gravity of the rivets. This is the problem of the eccentric riveted connection, which has been discussed in Chapter XV.

If the web is assumed to take part of the bending moment there will be an additional shear due to bending moment.

Rivets in the Flanges.—In Fig. 108, let $S =$ the shear in the girder at the given section, $h' =$ distance between rivet lines, $p =$ the pitch of the rivets, and $r =$ the resistance of one rivet (r is usually the safe bearing on the rivet in the web).

Then taking moments about the lower right hand rivet, we have

$$Sp = rh', \text{ and } p = rh' \div S \qquad (81)$$

Where the rivets are in double rows as shown in (d), the distance h' is taken as a mean of the distances for the two lines.

The crane loads produce an additional shear in the rivets, (e) Fig. 108, which will now be investigated. We will assume that the rail distributes the load over a distance of 25 inches; this distance will be less for light rails and more for heavy rails. The maximum vertical shear on one rivet will be $Pp \div 25 = 0.04\,Pp$. The horizontal stress due to bending moment is $r = (Sp \div h')$ and the resultant stress from the two sources will be

$$r' = \sqrt{(0.04\,Pp)^2 + \left(\frac{Sp}{h'}\right)^2}$$

and solving for p

$$p = \sqrt{\frac{r'}{(0.04\,P)^2 + \left(\frac{S}{h'}\right)^2}} \qquad \text{see below} \qquad (82)$$

Crane Girders.—The maximum moments and shears in crane girders are found as explained in Chapter X. For small cranes I beam girders are commonly used, and are designed by the use of their moments of inertia. Plate girders are designed as previously described. In designing both rolled and plate girders care must be used to properly support the girder laterally.

In above equation (82) r' is indeterminate since p is unknown. p is the unknown we want. $r' = \sqrt{H_s^2 + V_s^2}$ $\frac{S}{h'} = H_s$ due to load on girder, r' V_s due to load on rail.

Hence using Lewis's notation $r' = \sqrt{.04P^2 + \left(\frac{S}{h'}\right)^2}$. Now value of rivet divided by r' gives pitch of rivets or unit spacing. The rivets are in double shear hence bearing value is usually larger as it is greater than shearing value.

To account for p in above Equation (82) as above preceded, we must note that p is taken for a unit length (1").

CHAPTER XVIII.

Corrugated Steel.

Introduction.—Corrugated steel is made from sheet steel of standard gages, and is either galvanized at the mill or is left black. The black corrugated steel is usually painted at the mill and is always painted after erection. Paint will not adhere well to the galvanized steel until after it has weathered unless a portion of the coating is removed by the application of an acid. The common standard for the gage of sheet steel in the United States is the United States Standard Gage, and this should be used in specifying the weight and thickness. The thickness and weights per square of 100 square feet, for black and galvanized sheet and corrugated steel are given in Table XV. The weights of the corrugated steel given in the table are for standard corrugations, approximately 2½ inches wide and ⅝ of an inch deep. If black sheet steel is painted, add about 2 lbs. per square.

TABLE XV.

Weight of flat, and corrugated steel sheets with 2½-inch corrugations.

Gage No.	Thickness in Inches	Weight per Square (100 sq. ft.)			
		Flat Sheets		Corrugated Sheets	
		Black	Galvanized	Black Painted	Galvanized
16	.0625	250	266	275	291
18	.0500	200	216	220	236
20	.0375	150	166	165	182
22	.0313	125	141	138	154
24	.0250	100	116	111	127
26	.0188	75	91	84	99
28	.0156	63	79	69	86

Corrugated steel is also made with corrugations 5, 3 and 1¼ inches wide approximately. Corrugated steel with corrugations 1¼

inches wide and ⅜ of an inch deep is frequently used for lining buildings. Corrugated steel with 1¼-inch corrugations weighs about 4 per cent more than steel of the same gage with 2½-inch corrugations. Corrugated sheets are commonly made from flat bessemer steel sheets, by rolling one corrugation at a time. Iron and open hearth steel corrugated sheets can be obtained, but are very hard to get and cost extra.

The standard sheets of corrugated steel with 2½-inch corrugations, are 28 inches wide before, and 26 inches wide after corrugating, and will cover a width of 24 inches with one corrugation side lap, and approximately 21½ inches with two corrugations side lap, (c) and (a) Fig. 109. Special corrugated steel can usually be obtained that will cover a width of 24 inches with 1½ corrugations side lap, (b). Corrugated steel should be laid with 6 inches end lap on the roof and 4 inches end lap on the sides of buildings.

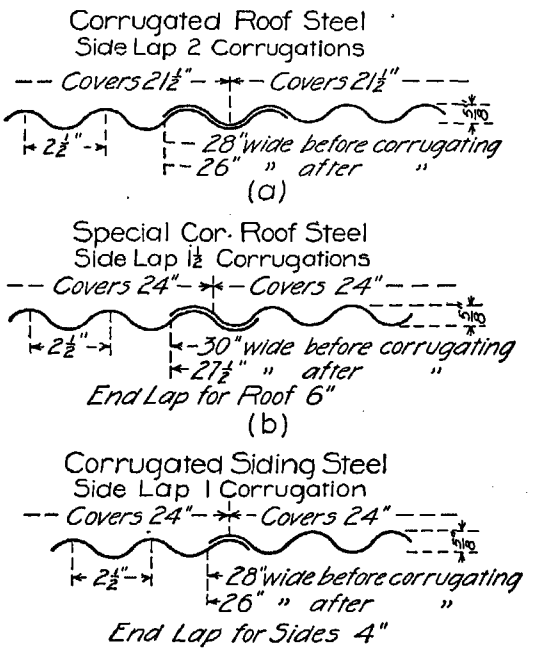


FIG. 109.

Stock lengths of corrugated steel sheets can be obtained from 5 to 10 feet, varying by one-half foot. Sheets of any length between 4 and 10 feet can usually be obtained directly from the mill without extra charge. Sheets from 4 8 to 5 inches long, cost from 1-10 to ½ cents per pound extra. Sheets from 10 to 12 feet long are very hard to obtain and cost extra. Sheets cannot be obtained longer than 12 feet. Stock lengths of sheets should be used whenever possible as odd lengths often delay the filling of the order. Bevel sheets should preferably be ordered in multiple lengths and should be cut in the field. Sheets to fit around windows and doors should be cut in the field; no part of a sheet less than ¼ the width of a full sheet should ever be used.

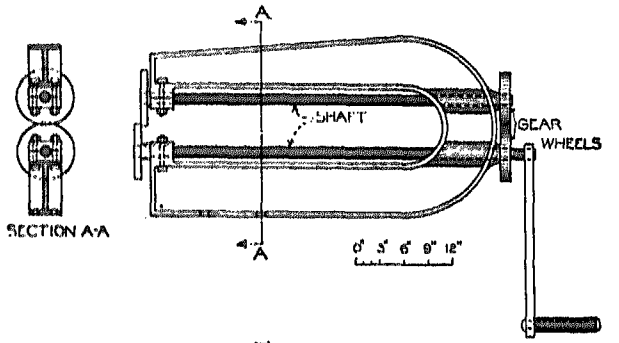


FIG. 110.

For cutting and splitting corrugated sheets in the field the rotary shear shown in Fig. 110 is invaluable. It will make square or bevel cuts, or will split sheets without denting the corrugations. The shear shown in Fig. 110 is one made by the Gillette-Herzog Mfg. Co., Minneapolis, Minn., and was used by the author in the erection of a steel stamp mill in Northern Michigan, while in the employ of the above named company. The shear is not on the market, but can be made in any ordinary machine shop at a comparatively small cost.

Fastening Corrugated Steel.—Where spiking strips are used, the corrugated steel is fastened with 8d barbed roofing nails ¾ to 2½ inches long, spaced 6 to 8 inches apart. The 2½-inch barbed nails should be used for nailing to spiking strips and to sheathing whenever possible. For weight of barbed roofing nails see Table XVI.

TABLE XVI.

NUMBER OF BARBED ROOFING NAILS IN ONE POUND.

Size	Length inches	Gage No.	No. in one lb.	Size	Length inches	Gage No.	No. in one lb.
4d	1½	13	339	20d	4	6	30
6d	2	12	205	30d	4½	5	23
8d	2½	10	96	40d	5	4	17
10d	3	9	63	50d	5½	3	13
12d	3½	8	52	60d	6	2	10
16d	3½	7	38				

The common methods of fastening corrugated steel directly to the purlins and girts are shown in Fig. 111. Nailing pieces should preferably be used where anti-condensation roofing, Fig. 127, is used, or where the sides are lined with corrugated steel. The clinch nail is prob-

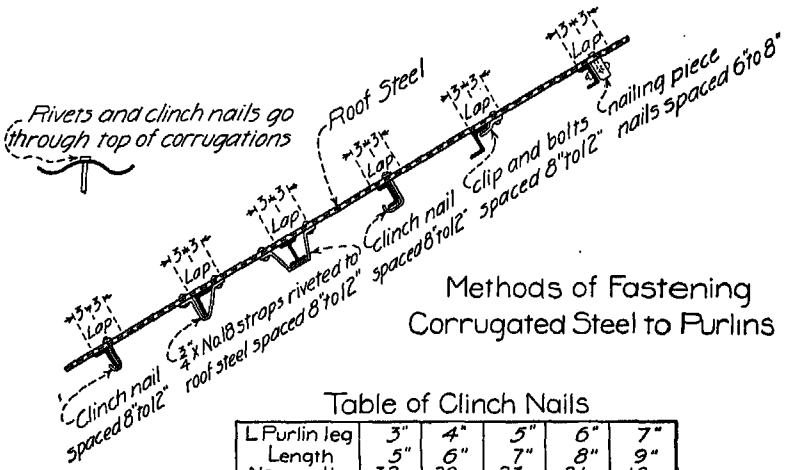


Table of Clinch Nails

L Purlin leg	3"	4"	5"	6"	7"
Length	5"	6"	7"	8"	9"
No. per lb.	32	29	23	21	18
C Purlin leg	3"	4"	5"	6"	7"
Length	6"	7"or8"	9"	10"	11"
No. per lb.	29	21	18	16	14

FIG. 111. METHODS OF FASTENING CORRUGATED STEEL TO PURLINS AND GIRTS.

ably the most satisfactory fastening for the usual conditions. The side laps are fastened together by means of copper or galvanized iron clos-

ing rivets, spaced about 8 to 12 inches apart on the roof and about 2 feet apart on the sides.

Clinch nails are made of ⅛ inch or No. 10 soft iron wire and are clinched around the purlin. The usual sizes and weights of clinch nails for different lengths of angle and channel purlins are given in Fig. 111. Care should be used in punching the holes in the corrugated steel for clinch nails and rivets to get them in the top of the corrugations and to avoid making the hole unnecessarily large. Clinch nails are spaced from 8 to 12 inches apart. Two clinch nails are usually furnished for each lineal foot of purlin and girt.

Straps are made of No. 18 gage steel, ¾ inches wide, and are

TABLE XVII.

NUMBER OF COPPER RIVETS IN ONE POUND.

Diameter Gage No.	Length of Rivets in Inches.													
	½	⅝	11/16	¾	13/16	⅞	15/16	1	1⅛	1¼	1⅜	1½	1⅝	1¾
3	70													
4	78													
5	85					64	60	53	48	46	44	39	36	32
6	180			105	100	96	90	74	68	61	56	54	50	46
7	368	211	180	171	160	150	140	132	110	97	91	79	72	63
8	417	266	248	227	200	172	157	147	136	116	100	93	88	71
9	600	365	336	261	248	228	220	184	169	156	133	124	113	99
10	820	411	376	336	305	257	249	223	206	180	162			
11	944	416	400	360	338	320								
12	1167	545	475	400	342	325	308	292	257	221	190			
13	1442	799	640	547	502	448	400	392	316					
14	1620	1040	995	816	784	616	550	528						
15	3512													

fastened with two 3-16-inch stove bolts ⅜ inches long. Straps are spaced 8 to 12 inches apart. One strap and two bolts are usually furnished for each lineal foot of purlin and girt. One bundle of hoop steel for making straps contains 400 lineal feet and weighs 50 lbs.

Clips are made of No. 16 gage steel, 1½" × 2½", and are fastened with two 3-16-inch stove bolts ½ inches long. Clips are spaced from 8 to 12 inches apart. One clip and two bolts are usually furnished for each lineal foot of purlin and girt.

Copper rivets weighing about 6 pounds per 1000 rivets have commonly been used for closing rivets; but galvanized iron rivets made of very soft wire and weighing about 7 pounds per 1000 rivets are fully as good, and cost 7 cents per pound in 1903 as compared with about 25 cents per pound for copper rivets. The weight of copper rivets is given in Table XVII.

Strength of Corrugated Steel.—The safe load per square foot for corrugated steel supported as a simple beam, for sheets with 2½-inch corrugations and of various gages is given in Fig. 112. This diagram is based on Rankine's formula

$$W = \frac{32}{15} \frac{S h b t}{l}$$

where W = safe load in lbs.;
S = working stress in lbs.;
h = depth of the corrugations in inches;
b = width of the sheet in inches;
t = thickness of the sheet in inches;
l = clear span in inches.

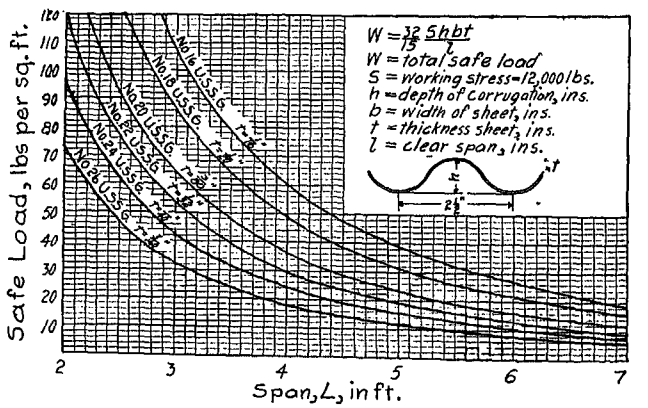


FIG. 112. SAFE UNIFORM LOAD IN POUNDS FOR CORRUGATED STEEL, FOR DIFFERENT SPANS IN FEET.

A summary of experiments to determine the strength of corrugated steel made by the author's assistant, Mr. Ralph H. Gage, is given in

Table XVIII. The coefficient C in column 8 depends on the angle that the metal makes with the horizontal axis and varies as follows: angle of 30°, $C = 0.278$; 45°, $C = 0.293$; 60°, $C = 0.312$, and for 90°, $C = 0.393$.

TABLE XVIII.

SUMMARY OF EXPERIMENTS TO DETERMINE THE STRENGTH OF CORRUGATED STEEL.*

1 No.	2 Width of Corrugations Ins.	3 Depth h Ins.	4 Thickness t Ins.	5 Angle of Metal with Axis	6 Span l Ins.	7 Tensile Strength lbs. per sq. in.	8 Gage's Formula $M = CShbt$ lbs.	9 Actual Brkng Load W lbs.	10 Rankine's Formula. $M = \frac{1}{12}Shbt$ lbs.
1	2.50	0.6025	.0588	39°11′	44.0	58,000	643	630	597
2	2.50	0.612	.0568	39°10′	44.0	58,000	632	630	587
3	2.50	0.625	.066	39°30′	44.0	58,000	745	720	692
4	2.50	0.606	.0655	40°42′	44.0	58,000	725	700	670
5	2.88	0.650	.0367	36° 0′	43.25	67,000	505	500	475
6	2.88	0.650	.0366	36° 0′	44.0	67,000	494	490	465
7	2.50	0.630	.0366	36° 0′	44.0	50,000	358	350	335
8	2.50	0.61	.0365	36° 0′	44.0	50,000	344	340	324
9	1.25	0.27	.0365	36° 0′	24.0	50,000	281	300	262
10	1.25	0.27	.0365	36° 0′	24.0	50,000	281	295	262
11	1.25	0.27	.0293	36° 0′	24.0	50,000	225	200	211
12	1.25	0.27	.0293	36° 0′	24.0	50,000	225	195	211
13	1.00	0.18	.0291	36° 0′	24.0	50,000	298	310	279
14	1.00	0.18	.0291	36° 0′	24.0	50,000	298	300	279
15	1.00	0.18	.026	36° 0′	24.0	50,000	266	280	250
16	1.00	0.18	.026	36° 0′	24.0	50,000	266	260	250

The actual breaking load agrees in most cases more closely with Gage's formula than with Rankine's, although the latter is more often on the safe side.

Purlins are commonly spaced for a safe load of 30 lbs. per square foot as given in Fig. 112; if the purlins are spaced farther apart than this, the steel will deflect a dangerous amount when walked on, and will leak snow and rain. Girts should be spaced for a safe load of about 25 lbs. per square foot. From an inspection of Fig. 112, it is evident that corrugated steel lighter than No. 24 is of little use for mill buildings.

*For details of experiments see article by Ralph H. Gage, in the Technograph, No. 17.

Corrugated steel of No. 26 or 28 gage is so thin that it soon rusts out and should never be used unless for lining cheap buildings.

Corrugated Steel Details.—*Ridge Roll.*—The ridge roll most commonly used is made from No. 24 flat steel, and has a 2½-inch roll and 6-inch aprons. It comes in 96-inch lengths and should be laid with 3 inches end lap. Plain and corrugated ridge roll are used (see Fig.

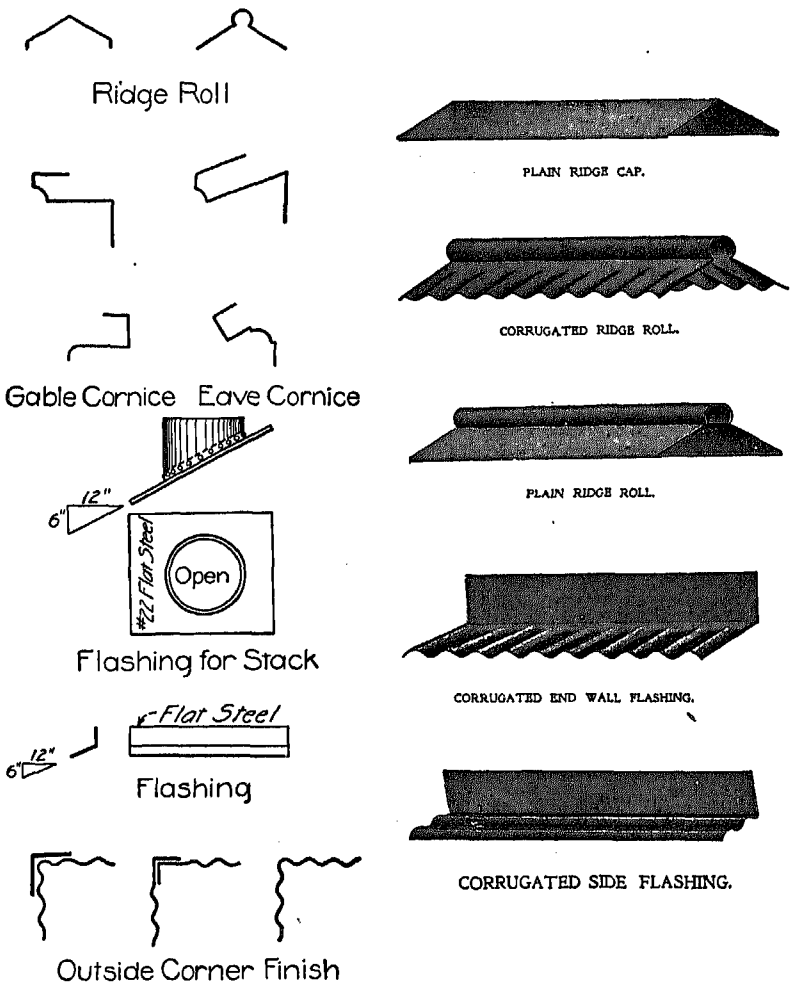
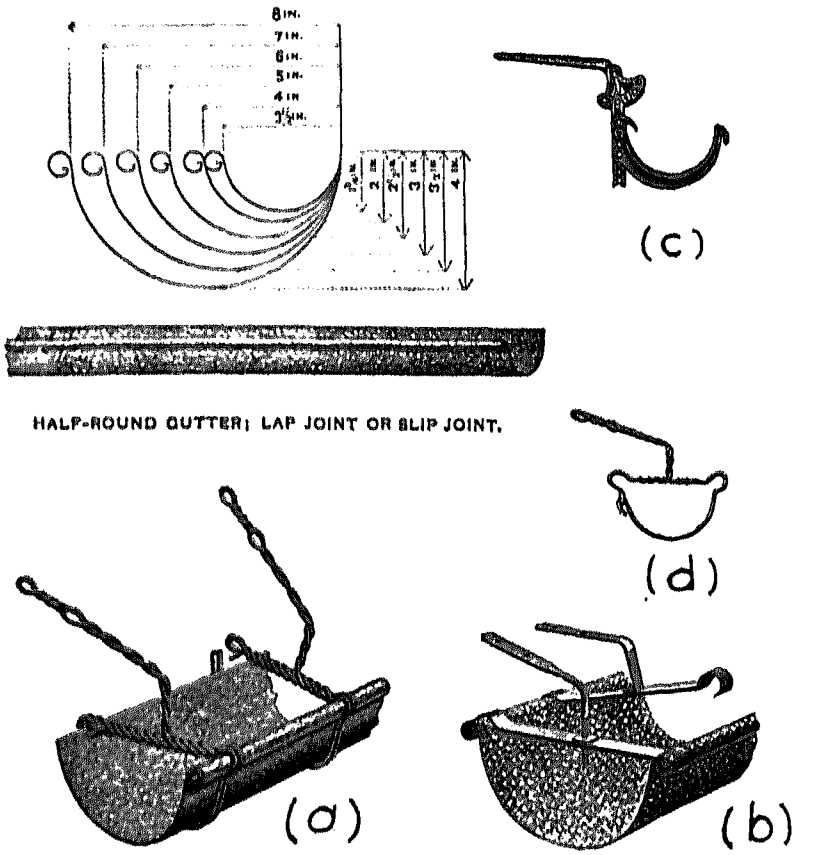


FIG. 113.

113). Ridge roll is fastened with rivets or nails spaced 6 to 8 inches apart.

Flashing.—Flashing is used where the roof changes slope, around chimneys and openings in the roof, and over windows and doors, and should be of sufficient dimensions and so arranged that at least 3 inches vertical height is obtained between the edge of the flashing and the end of the corrugated steel roofing. Vertical and horizontal seams of all flashing should be closely riveted. Flashing is made from flat sheets

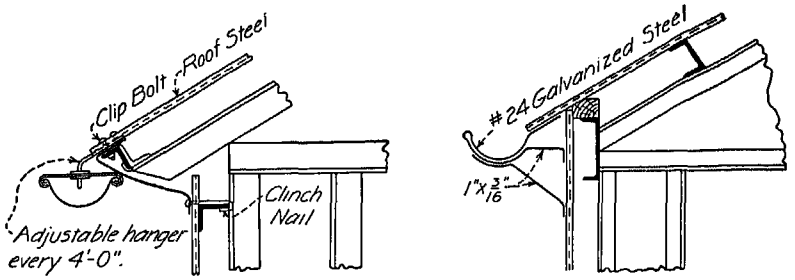


EAVES TROUGH HANGERS

FIG. 114.

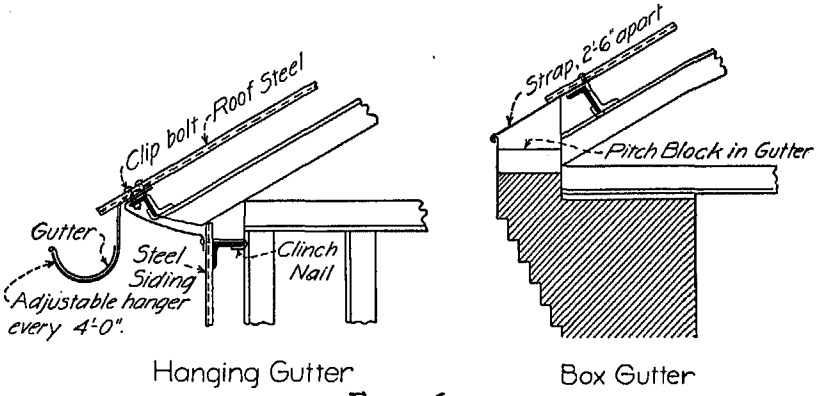
of the same gage as the corrugated steel, and can be obtained up to 96 inches in length. Flashing is made both plain and corrugated (see Fig. 113).

Corner Finish.—Corner finish is made in various ways, three of which are shown in Fig. 113. Other methods are shown on the succeeding pages.



Hanging Gutter Hanging Gutter

FIG. 115.



Hanging Gutter Box Gutter

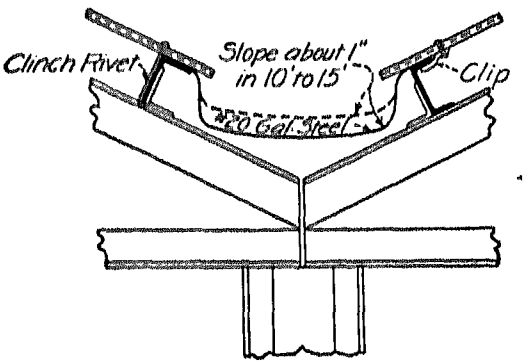
FIG. 116.

Gutters and Conductors.—Gutters for eaves are ordinarily made from No. 24, and valley gutters from No. 20 galvanized steel. Gutters may be obtained in even foot lengths up to 10 feet, and should have 4-inch end laps. Special flat sheets up to 42 inches in width can be obtained for making gutters and details.

The common sizes of half round gutters made by the Garry Iron and Steel Roofing Co., Cleveland, Ohio, are shown in Fig. 114. Two common forms of adjustable hangers are shown in (a) and (b) in Fig. 114.

Two forms of hanging gutters are shown in Fig. 115 and one form of a hanging, and a box gutter used with brick walls are shown in Fig. 116.

A standard form of valley gutter is shown in Fig. 117. Extreme care should be used in making valley gutters to see that the sides are carried well up, and that the laps are well soldered.



Valley Gutter

FIG. 117.

Conductors are made plain round or square, and corrugated round or square. Corrugated conductors are to be preferred to plain conductors for the reason that they will give when the ice freezes inside of them, and will not burst as the others often do. Common sizes of round pipe are 2", 3", 4", 5", and 6" diameter. Common sizes of square pipe are 1¼" × 2¼", 2⅜" × 3¼", 2¾" × 4¼" and 3¾" × 5", equal to 2", 3" and 4" round pipe, respectively. Conductor pipes are fastened with hooks or by means of wire.

Design of Gutters and Conductors.—The specifications of the American Bridge Company for the design of gutters and conductors are as follows:

Span of roof.	Gutter.	Conductor.
up to 50′	6″	4″ every 40′
50′ to 70′	7″	5″ " 40′
70′ to 100′	8″	5″ " 40′

Hanging gutters should have a slope of at least 1 inch to 15 feet.

The diagram in Fig. 118 for the design of gutters and conductors was described in Engineering News, April 17, 1902, by Mr. Emmett Steece, Assoc. M. A. Soc. C. E., City Engineer of Burlington, Iowa, as follows:—

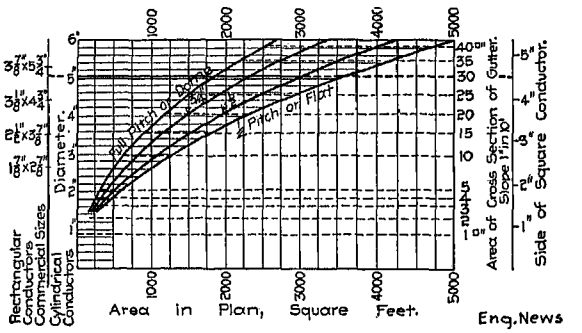


FIG. 118.

"The curves are for ¼ pitch or flat roofs, to full pitch or domes. The areas are reduced to plan as shown. The minimum sizes of circular and commercial rectangular conductors are given on the left side of the diagram and the sizes and the minimum cross-sectional areas of square gutters are given on the right hand side.

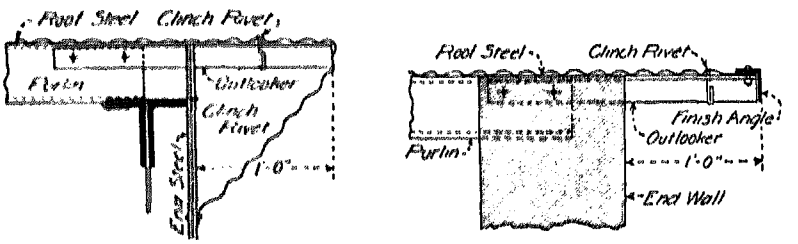
To use the diagram: Assume an area of roof, say 30 × 100 ft., or 3000 sq. ft., ½ pitch and one conductor for the whole area. Note the intersection of the vertical over area 3000 and the curve of ½ pitch; following thence the horizontal line to the left it strikes a diameter of 5 ins. for circular, or over 3⅛ × 4¾ ins. for commercial size. The next larger size would be used. The minimum cross-sectional area of gutters is shown on the right to be about 30 sq. ins., and the side of a square conductor about 4.5 ins."

This diagram was based on a maximum rainfall of 1.98 inches per hour.

English practice is as follows: Rain-water or down-pipes should have a bore or internal area of at least one square inch for every 60 square feet of roof surface in temperate climates, and about 35 square feet in tropical climates. They should be placed not more than 20 feet apart, and should have gutters not less in width than twice the diameter of the pipe.

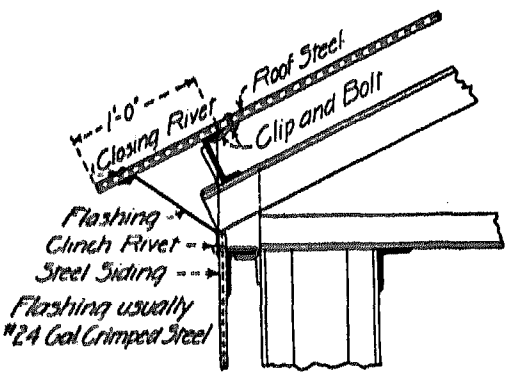
The practice among American architects is to provide about one square inch of conductor area for each 75 square feet of roof surface; no conductor less than 2 inches in diameter being used in any case.

Cornice.—There are many methods of finishing the gables and eaves of buildings. A gable finish for a steel end, and for a brick end



Gable Finish for Steel End — Gable Finish with Brick Wall

FIG. 119.



Flashed Finish

FIG. 120.

as used by the American Bridge Company, are shown in Fig. 119. The steel end may have a cornice made by bending the corrugated steel as shown, or a molded cornice.

The flashed finish shown in Fig. 120, is used by the American Bridge Company; it is quite effective and gives a very neat appearance. The corrugated steel siding should preferably be carried up to the roof steel.

The cornice and ridge finish shown in Fig. 121, designed by Mr. H. A. Fitch, Minneapolis, Minn., is very neat, efficient and economical.

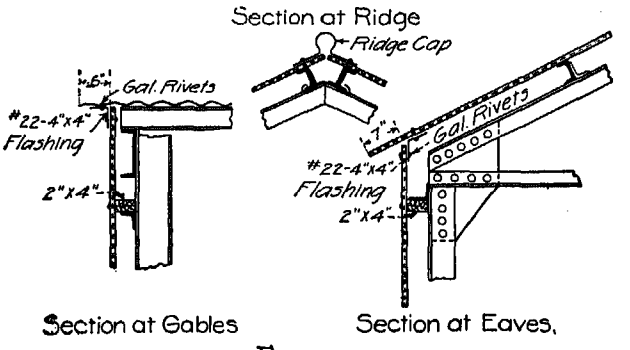


FIG. 121.

The galvanized rivets are much cheaper than copper rivets, and are preferred by many to the copper rivets. The detail shown was for a small dry house in which the eave strut was omitted.

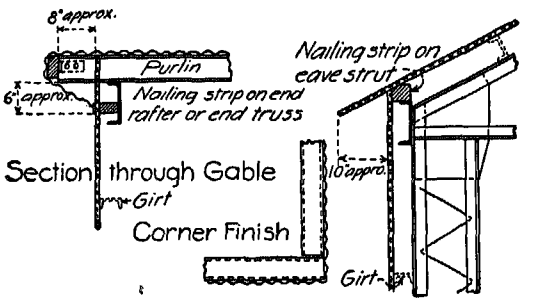


FIG. 122.

In Fig. 122, the eave cornice is made by simply extending the roofing steel, while the gable cornice is made by bending a sheet of corrugated steel over the ends of the purlins and nailing to wooden strips as shown.

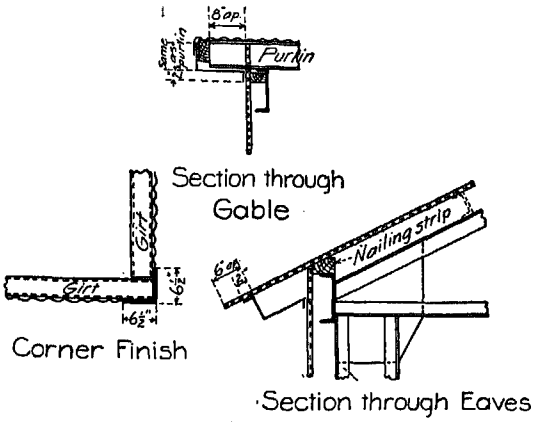


FIG. 123.

Sheets heavier than No. 22 should not be bent in the field. The corner finish is made by bending a sheet of corrugated steel.

In Fig. 123, the eave and gable cornice are made of plain flat steel bent in the shop as shown. The eave cornice is made to mitre with the gable cornice, thus giving a neat finish at the corner. The corner finish is made by using sheets at the corners in which one-half is left plain.

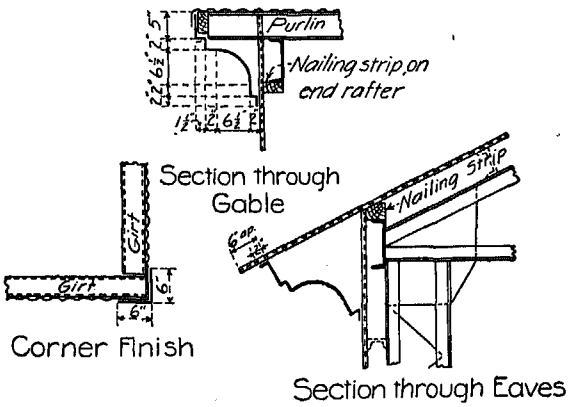


FIG. 124.

In Fig. 124, the eave strut and gable cornice are molded. The two cornices are so made as to mitre at the corners, the mitres being made in the field. A plain corner cap is put on as shown, after bending the corrugated steel around the corner.

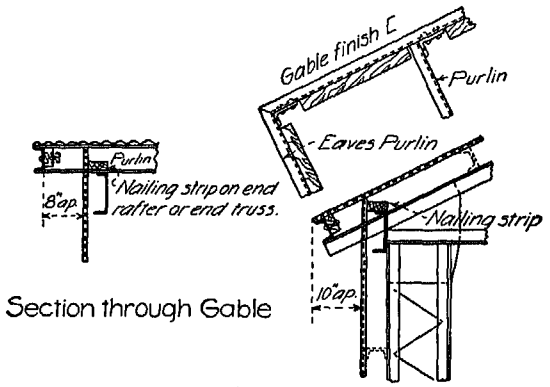


FIG. 125.

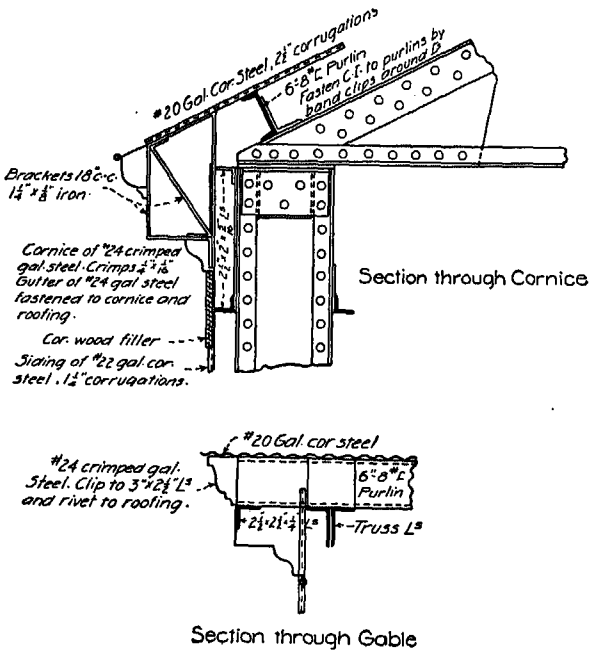
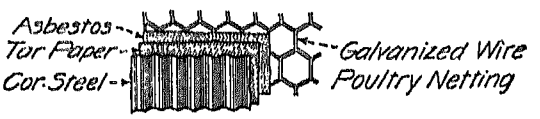


FIG. 126.

In Fig. 125, an eave purlin is used and a channel is placed along the ends of the purlins. Spiking strips should always be used as shown, and the eave purlin should be fastened to the rafter by means of angle clips.

The finish shown in Fig. 126, was used by the U. S. Government and needs no explanation.

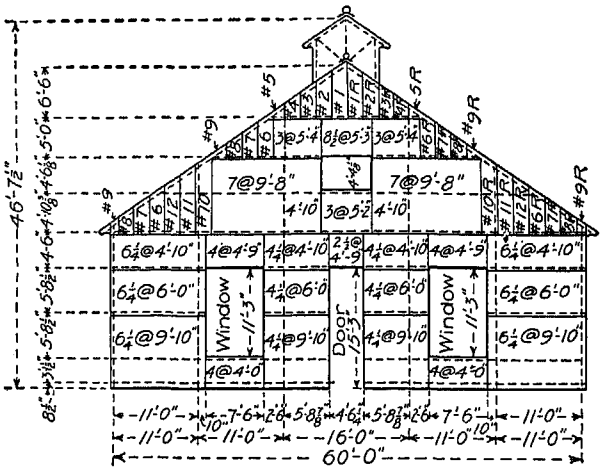
Anti-condensation Roofing.—To prevent the condensation of moisture on the inner surface of a steel roof, and the resulting dripping, the *anti-condensation roofing* shown in Fig. 127 and in Fig. 129 is frequently used. The usual method of constructing this roofing is as follows: Galvanized wire poultry netting is fastened to one eave purlin



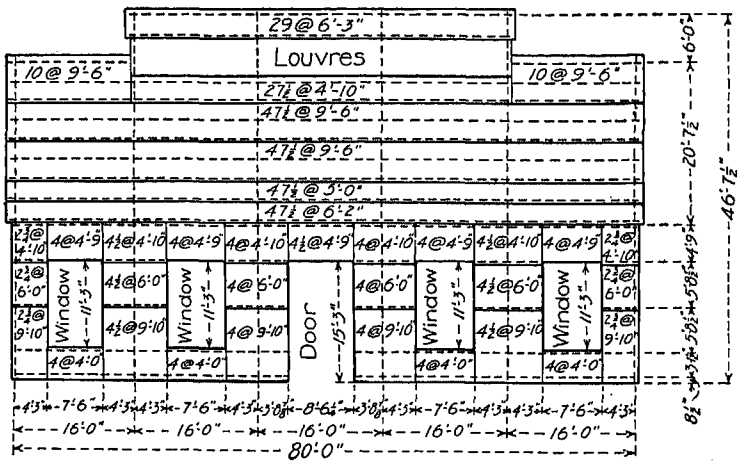
Anti-Condensation Roofing
FIG. 127.

and is passed over the ridge, stretched tight and fastened to the other eave purlin. The edges of the wire are woven together, and the netting is fastened to the spiking strips, where used, by means of small staples. On the netting are laid one or two layers of asbestos paper 1-16-inch thick, and sometimes one or two layers of tar paper. The corrugated steel is then fastened to the purlins in the usual way. Stove bolts, 3-16" diameter, with 1 × ⅛ × 4-inch plate washers on lower side, are used for fastening the side laps together and for supporting the lining (see Fig. 129). The author would recommend that purlins be spaced one-half the usual distance where anti-condensation lining is used; the stove bolts could then be omitted. Asbestos paper 1-16-inch thick comes in rolls, and weighs about 32 pounds per square of 100 square feet. Galvanized poultry netting comes in rolls 60 inches wide and weighs about 10 pounds per square.

The corrugated steel used with anti-condensation roofing should never be less than No. 22, and the purlins should be spaced for not less

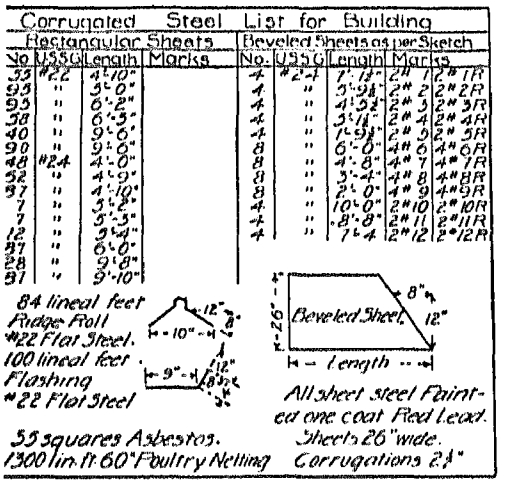


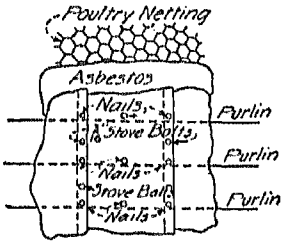
End Elevation



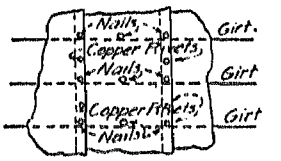
Side Elevation

FIG. 128. CORRUGATED STEEL PLANS FOR A TRANSFORMER BUILDING.

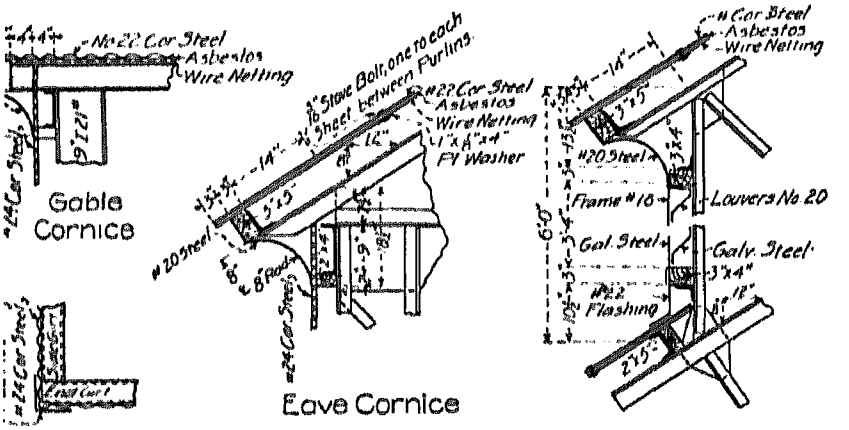




Method of Fastening Steel and Lining on Roof



Method of Fastening Steel on the Sides



ish at Corner

G. 120. CORRUGATED STEEL LIST AND DETAILS FOR TRANSFORMER BUILDING.

than 30 pounds per square foot. A less substantial roof will not usually be satisfactory.

An engine house with anti-condensation lining on the roof and sides has been in use in the Lake Superior copper country for several years, and has been altogether satisfactory under trying conditions. The covering and lining of roof and sides are fastened by clinch nails to angle purlins and girts spaced about two feet apart.

A transformer building designed by the author and built by the Gillette-Herzog Mfg. Co., at East Helena, Montana, has anti-condensation lining on the roof as shown in Fig. 129, and is lined on the sides with one layer of asbestos paper, and 1¼-inch No. 26 corrugated steel. The black framework, the red side lining, and white roof lining made a very pleasing interior. This building after several years is giving entire satisfaction.

Corrugated Steel Plans.—The shop plans, list of steel and details of the corrugated steel for a mill building are shown in Fig. 128 and Fig. 129 (for the general plans and a detailed estimate of this building see Chapter XXVIII). Corrugated steel sheets should be ordered to cover three purlins or girts if possible. Bevel sheets should be ordered by number, and sheets should be split and reentrant cuts should be made in the field. All sheets should be plainly marked with the number or length. Sheets No. 22 or lighter can be bent in the field, heavier metal should always be bent at the mill. In preliminary estimates of corrugated steel allow 25 per cent for laps where two corrugations side lap and 6 inches end lap are required, and 15 per cent for laps where one corrugation side lap and 4 inches end lap are required.

Cost of Corrugated Steel.—Galvanized steel in 1903 is quoted at about 75 per cent off the standard list, f. o. b. Pittsburg; list price of flat galvanized steel being as follows:

No. 10 to 16 inclusive	12c.	per lb.
No. 17 to 21 inclusive	13c.	" "
No. 23 to 24 inclusive	14c.	" "
No. 25 to 26 inclusive	15c.	" "
No. 27	16c.	" "

The net cost of corrugated galvanized steel is found by adding .05c. per pound to the net cost of flat galvanized sheets.

The standard card of extras used in 1903 is given below. These extras are to be added to the net price of flat black or galvanized sheets to obtain the cost. These extras are not subject to discount.

CARD OF EXTRAS FOR BLACK OR GALVANIZED SHEETS.

For corrugating	.05c.	per lb.
For painting with red oxide	.10c.	" "
For painting with Dixon's graphite	.20c.	" "
For painting with Goheen's carbonizing coating	.30c.	" "
For all trimmings, etc., flashings, ridge caps, and louvres	1.00c.	" "
For flat sheets rolled from reworked muck bar	.50c.	" "
For sheets rolled from iron scrap mixture	.25c.	" "
For arches	.25c.	" "

Black corrugated steel in 1903 is quoted about as follows, f. o. b. Pittsburg:

No. 16 to 18 inclusive	2.2c.	per lb.
No. 20 to 22 inclusive	2.6c.	" "
No. 24 to 26 inclusive	2.7c.	" "

CHAPTER XIX.

Roof Coverings.

Introduction.—Mill buildings are covered with corrug supported directly on the purlins; by slate or tile supporte purlins; or by corrugated steel, slate, tile, shingles, gravel composition roof, or some one of the various patented roo ported on sheathing. The sheathing is commonly made o thickness of planks, 1 to 3 inches thick. The planks are some in two thicknesses with a layer of lime mortar between the l protection against fire. In fireproof buildings the sheathin monly made of reinforced concrete constructed as described i XX. Concrete slabs are sometimes used for a roof covering that case supported directly by the purlins, and sometimes as ing for a slate or tile roof.

The roofs of smelters, foundries, steel mills, mine stru similar structures are commonly covered with corrugated ste the buildings are to be heated or where a more substantial ering is desired slate, tile, tin or a good grade of composition is used, or the roof is made of reinforced concrete. For t and for temporary roofs a cheap composition roofing is comm The following coverings will be described in the order given ated steel, slate, tile, tin, sheet steel, gravel, slag, asphalt, sl also the patent roofings; asbestos, Carey's, Granite, Ruberoi roinclave. The construction of reinforced concrete roofi scribed in Chapter XX.

Corrugated Steel Roofing.—Corrugated steel roofing plank sheathing or is supported directly on the purlins as d Chapter XVIII. For the cost of erecting corrugated steel r Chapter XXVIII.

Corrugated steel roofing should be kept well painted with a good paint. Where the roofing is exposed to the action of corrosive gases as in the roof of a smelter reducing sulphur ores, ordinary red lead or iron oxide paint is practically worthless as a protective coating; better results being obtained with graphite and asphalt paints. Graphite paint has been quite extensively used for painting corrugated steel in the Butte, Mont., district. The corrosion of corrugated steel is sometimes very rapid. In 1898 the author saw at the Trail Smelter, Trail, B. C., a corrugated steel roof made of No. 22 corrugated steel and painted with oxide of iron paint that had corroded so badly in one year that one could stick his finger through it as easily as through brown paper. The climate in that locality is moist and the smelter was used for reducing sulphur ores. Galvanized corrugated steel is quite extensively used in the Lake Superior district.

Slate Roofing.—There are many varieties of roofing slate, among which the Brownville and Monson slates of Maine, and the Bangor and Peach Bottom slates of Pennsylvania are well known and are of

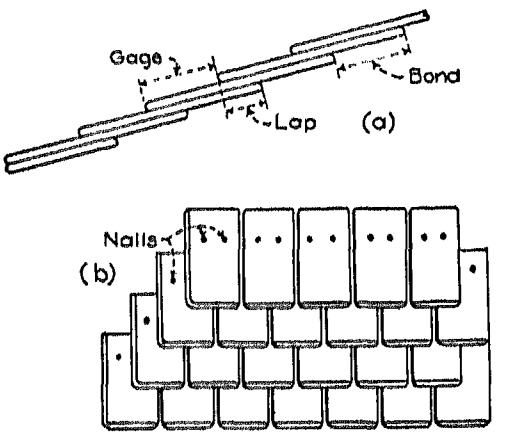


FIG. 130.

excellent quality. Besides the characteristic slaty color, green, purple, red and variegated roofing slates may be obtained. The best quality of slate has a glistening semi-metallic appearance. Slate with a dull

earthy appearance will absorb water and is liable to be destroyed by the frost.

Roofing slates are usually made from 1/8 to 1/4 inches thick; 3-16-inch being a very common thickness. Slates vary in size from 6″ × 12″ to 24″ × 44″; the sizes varying from 6″ × 12″ to 12″ × 18″ being the most common.

Slates are laid like shingles as shown in Fig. 130. The lap most commonly used is 3 inches; where less than the minimum pitch of 1/4 is used the lap should be increased.

The number of slates of different sizes required for one square of 100 square feet of roof for a 3-inch lap are given in Table XIX.

The weight of slates of the various lengths and thicknesses required for one square of roofing, using a 3-inch lap is given in Table XX. The weight of slate is about 174 pounds per cubic foot.

The weight of slate per superficial square foot for different thicknesses is given in Table XXI.

The minimum pitch recommended for a slate roof is 1/4; but even with steeper slopes the rain and snow may be driven under the edges of the slates by the wind. This can be prevented by laying the slates in slater's cement. Cemented joints should always be used around eaves, ridges and chimneys.

Slates are commonly laid on plank sheathing. The sheathing should be strong enough to prevent deflections that will break the slate, and should be tongued and grooved, or shiplapped, and dressed on the upper surface. Concrete sheathing reinforced with wire lath or expanded metal is now being used quite extensively for slate and tile roofs, and makes a fireproof roof. Tar roofing felt laid between the slates and the sheathing assists materially in making the roof waterproof, and prevents breakage when the roof is walked on. The use of rubber-soled shoes by the workmen will materially reduce the breakage caused by walking on the roof. Roofing slates may also be supported directly on laths or sub-purlins. The details of this method are practically the same as for tile roofing, which see.

TABLE XIX.

NUMBER OF ROOFING SLATES REQUIRED TO LAY ONE SQUARE OF ROOF WITH 3-INCH LAP.

Size in Inches.	No. of Slate in Square.	Size in Inches.	No of Slate in Square.	Size in Inches.	No. of Slate in Square.
6×12	533	8×16	277	12×20	141
7 12	457	9 16	246	14 20	121
8 12	400	10 16	221	11 22	137
9 12	355	12 16	184	12 22	126
10 12	320	9 18	213	14 22	108
12 12	266	10 18	192	12 24	114
7 14	374	11 18	174	14 24	98
8 14	327	12 18	160	16 24	86
9 14	291	14 18	137	14 26	89
10 14	261	10 20	169	16 26	78
12 14	218	11 20	154		

TABLE XX.

THE WEIGHT OF SLATE REQUIRED FOR ONE SQUARE OF ROOF.

Length in Inches.	Weight in pounds, per square, for the thickness.							
	1/8"	3/16"	1/4"	5/16"	1/2"	5/8"	3/4"	1"
12	483	724	967	1450	1936	2419	2902	3872
14	460	688	920	1370	1842	2301	2760	3683
16	445	667	890	1336	1784	2229	2670	3567
18	431	650	869	1303	1740	2174	2607	3480
20	425	637	851	1276	1704	2129	2553	3408
22	418	626	836	1254	1675	2093	2508	3350
24	412	617	825	1238	1653	2066	2478	3306
26	407	610	815	1222	1631	2039	2445	3263

TABLE XXI.

WEIGHT OF SLATE PER SQUARE FOOT.

Thickness—in.	1/8	3/16	1/4	5/16	1/2	5/8	3/4	1
Weight—lbs.	1.81	2.71	3.62	5.43	7.25	9.06	10.87	14.5

When roofing slates are laid on sheathing they are fastened by two nails, one in each upper corner. When supported directly on sub-purlins the slates are fastened by copper or composition wire. Galvanized and tinned steel nails, copper, composition and zinc slate roofing nails are used. Where the roof is to be exposed to corrosive gases copper, composition or zinc nails should be used.

Slate roofs when made from first class slates well laid have been known to last 50 years. When poorly put on or when an inferior quality of slate is used slate roofs are comparatively short-lived. Slates are easily broken by walking over the roof and are sometimes broken by hailstones. Slate roofing is fireproof as far as sparks are concerned, but the slates will crack and disintegrate when exposed to heat. Local conditions have much to do with the life of slate roofs; an ordinary life being from 25 to 30 years.

First class slate 3-16 to ¼ inches thick may ordinarily be obtained f. o. b. at the quarry for from $5.00 to $7.00 per square; common slate for from $2.00 to $4.00 per square; while extra fine slate may cost from $10.00 to $12.00 per square.

An experienced roofer can lay from 1½ to 2 squares of slate in a day of 10 hours. In 1903 slater's supplies were quoted as follows: Galvanized iron nails, 2½ to 3 cents per lb.; copper nails, 20 cents per lb.; zinc nails, 10 cents per lb.; slater's felt, 70 to 75 cents per roll of 500 square feet; two-ply tar roofing felt, 75 cents per square; slater's cement in 10-lb. kegs, 10 cents per lb.

Trautwine gives the cost of slate roofs as $7.00 per square and upwards. The costs of slate roofs per square is given in the reports of the Association of Railway Superintendents of Bridges and Buildings, as follows: New England, $9.00 to $12.00; New York, $9.00 to $10.00; Virginia $4.10 to $5.00; California, $10.00 to $10.50.

Tile Roofing.—Baked clay or terra-cotta roofing tiles are made in many forms and sizes. Plain roofing tiles are usually 10½ inches long, 6¼ inches wide and ⅝ inches thick; weigh from 2 to 2½ pounds each and lay one-half to the weather. There are many other forms of

tile among which book tile, Spanish tile, pan tile and Ludowici tile are well known. Tiles are also made of glass and are used in the place of skylights.

Tiles may be laid (1) on plank sheathing, (2) on concrete and expanded metal or wire lath sheathing, or (3) may be supported directly on angle sub-purlins as shown in Fig. 87. Tiles are laid on sheathing in the same manner as slates.

The roof shown in Fig. 87 was constructed as follows: Terra-cotta tiles, manufactured by the Ludowici Roofing Tile Co., Chicago, Ill., were laid directly on the angle sub-purlins, every fourth tile being secured to the angle sub-purlins by a piece of copper wire. The tiles were interlocking, requiring no cement except in exceptional cases. The tiles were 9 × 16 inches in size; 135 being sufficient to lay a square of 100 square feet of roof. These tiles weigh from 750 to 800 lbs. per square, and cost about $6.00 per square at the factory. Skylights in this roof were made by substituting glass tiles for the terra-cotta tiles. This and similar tile has been used in this manner on a large number of mills and train sheds with excellent results.

Tile roofs laid without sheathing do not ordinarily condense the steam on the inner surface of the roof unless the tiles are glazed, although several cases have been brought to the author's attention where the condensation has caused trouble with tile roofs made of porous tiles. Anti-condensation roof lining should be used where there is danger of excessive sweating, or a porous tile should be used that is known to be non-sweating. The cost of tile roofing varies so much that general costs are practically worthless. The reports of the Association of Railway Superintendents of Bridges and Buildings give the cost in New England as from $30.00 to $33.00 per square.

Tin Roofs.—Tin plates are made by coating flat iron or steel sheets with tin, or with a mixture of lead and tin. The former is called "bright" tin plate and the latter "terne" plate. Terne plates should not be used where the roof will be subjected to the action of corrosive gases for the reason that the lead coating is rapidly destroyed. Plates are

covered with tin (1) by the dipping process in which the plates are pickled in dilute sulphuric acid, annealed, again pickled, dipped in palm oil and then in a bath of molten tin or tin and lead; or (2) by the roller process in which the plates are run through rolls working in a large vessel containing oil, immediately after being dipped. The latter method gives the better results.

Two sizes of tin plates are in common use, 14" × 20" and 20" × 28", the latter size being most used. Tin sheets are made in several thicknesses, the IC, or No. 29 gage weighing 8 ounces to the square foot, and the IX, or No. 27 gage weighing 10 ounces to the square foot, being the most used. The standard weight of a box of 112 sheets, 14 × 20 size is 108 pounds for IC plate, and 136 pounds for IX plate. Boxes containing imperfect sheets or "wasters" are marked ICW or IXW. Every sheet should be stamped with the name of the brand and the thickness.

The value of tin roofing depends upon the amount of tin used in coating and the uniformity with which the iron has been coated. The amount of tin used varies from 8 to 47 pounds for a box of 20 × 28 size containing 112 sheets.

Tin roofing is laid (1) with a flat seam, or (2) with a standing seam. In the former method the sheets of tin are locked into each other at the edges, the seam is flattened and fastened with tin cleats or is nailed firmly and is soldered water tight. Rosin is the best flux for soldering, although some tinners recommend the use of diluted chloride of zinc. For flat roofs the tin should be locked and soldered at all joints, and should be secured by tin cleats and not by nails. For steep roofs the tin is commonly put on with standing seams, not soldered, running with the pitch of the roof, and with cross-seams double locked and soldered. One or two layers of tar paper should be placed betwen the sheathing and the tin.

In painting tin all traces of grease and rosin should be removed, benzine or gasoline being excellent for this purpose. A paint composed of 10 pounds venetian red and one pound red lead to one gallon of

pure linseed oil is recommended. The under side of the sheets should be painted before laying. Tin roofs should be painted every two or three years. If kept well painted a tin roof should last 25 to 30 years.

For flat seam roofing, using ½-inch locks, a box of 14 × 20 tin will cover 192 square feet, and for standing seam, using ⅜-inch locks and turning 1¼ and 1½-inch edges, making 1-inch standing seams, it will lay 168 square feet. For flat seam roofing, using ½-inch locks, a box of 20 × 28 tin will lay about 399 square feet, and for standing seam, using ⅜-inch locks and turning 1¼ and 1½-inch edges, making 1-inch standing seams, it will lay about 365 square feet.

Current prices in 1903 for tin in small quantities were about as follows:

American Charcoal Plates:

IC, 14 × 20.....$5.50 to $6.50 per box of 112 sheets;

IX, 14 × 20.....$6.60 to $8.25 per box of 112 sheets.

American Coke Plates, Bessemer:

IC, 14 × 20.....$4.70 to $4.80 per box of 112 sheets;

IX, 14 × 20.....$6.60 to $8.25 per box of 112 sheets.

American Terne Plates:

IC, 20 × 28.....$ 9.50;

IX, 20 × 28.....$11.50.

Two good workmen can put on and paint from 2½ to 3 squares of tin roofing in 8 hours. Tin roofs cost from $7.00 to $11.00 or $12.00 per square depending upon the specifications and the cost of labor.

Sheet Steel Roofing.—Sheet steel roofing is sold in sheets 28 inches wide and from 4 to 12 feet long, or in rolls 26 inches wide and about 50 feet long. It is commonly laid with vertical standing seams and horizontal flat seams; tin cleats from 12 to 15 inches apart being nailed to the plank sheathing and locked into the seams. Sheet steel plates are also made with standing crimped seams near the edges, which are nailed to V-shaped sticks; the horizontal seams being made by lapping about 6 inches.

Care should be used in laying sheet steel roofing to see that it does not come in contact with materials containing acids, and it should be kept well painted. The weight of flat steel of different gages is given in Table XV. Nos. 26 and 28 gage sheets are commonly used for sheet steel roofing. No. 26 black sheet steel was quoted in 1903 at about $3.20 per 100 pounds, and No. 26 galvanized sheet steel at about $4.00 per 100 pounds in small lots. Sheet steel roofing can be laid at a somewhat less cost than tin roofing.

Gravel Roofing.—Gravel roofing is made by laying and firmly nailing several layers of roofing felt on sheathing so as to break joints from 9 to 12 inches; the laps are mopped and cemented together with roofing cement or tar, and finally the entire surface is covered with a good coating of hot cement or tar. The cement or tar should not be hot enough to injure the fibre of the felt. While the cement or tar is still hot the surface of the roof is covered with a layer of clean gravel that has been screened through a ⅝-inch mesh. It requires from 8 to 10 gallons of tar or cement and about ⅙ of a yard of gravel per square of 100 square feet of roof. When the roof is to be subjected to the action of corrosive gases it should be flashed with copper or composition, or the flashing may be made of felt. The number of layers of felt varies with the conditions, but should never be less than four (4-ply).

The details of laying gravel roofs differ and it is impossible to do more than give a few standard specifications. The following specifications are about standard in the West. In writing specifications for four-ply gravel roofing omit one layer of roofing felt in the specifications for five-ply gravel roofing. Three-ply roofing is sometimes used for temporary structures.

Five (5) *Ply Wool Felt, Composition and Gravel Roof.*—First cover the sheathing boards with one (1) layer of dry felt and over this put four (4) thicknesses of wool roofing felt, weighing not less than fifteen (15) pounds (single thickness) to the square of one hundred (100) feet. This felt to be smoothly and evenly laid and well cemented together the full width of the lap, not less than nine (9) inches between each layer, with best roofing cement or refined tar, using not less than

one hundred (100) pounds of roofing cement or tar to the square of one hundred (100) feet. All joinings along walls and around openings to be carefully made. The roof to be then covered with a heavy coating of roofing cement or tar and screened gravel, not less than one (1) cubic yard of gravel to six hundred (600) square feet, gravel to be screened through ⅝-inch mesh and free from sand and loam. All walls and openings to be flashed. All roofing cement and tar is to be applied hot.

Six (6) Ply Cap Sheet Wool Felt, Composition and Gravel Roof.—First cover the sheathing boards with one (1) layer of dry felt and over this put four (4) thicknesses of wool roofing felt, weighing not less than fifteen (15) pounds (single thickness) to the square of one hundred (100) feet. This felt to be smoothly and evenly laid and well cemented together the full width of the lap, not less than nine (9) inches between each layer, with best roofing cement or refined tar, using not less than one hundred and twenty (120) pounds of roofing cement or tar to the square of one hundred (100) feet. The entire surface then to be mopped over with roofing cement or tar and a cap sheet of wool felt applied. All joinings along the walls and around the openings to be carefully made. The roof to be then covered with a heavy coating of roofing cement or tar and screened gravel, not less than one (1) cubic yard of gravel to six hundred (600) square feet, gravel to be screened through ⅝-inch mesh and free from sand and loam. All walls and openings to be flashed. All roofing cement and tar shall be applied hot.

Six (6) Ply Combined Flax and Wool Felt, Composition and Gravel Roof.—First cover the sheathing boards with one (1) layer of dry felt and over this put one (1) layer of flax felt and three thicknesses of wool roofing felt, weighing not less than fifteen (15) pounds (single thickness) to the square of one hundred (100) feet. This felt to be smoothly and evenly laid and well cemented together the full width of the lap, not less than eleven (11) inches between each layer, with best roofing cement or refined tar, using not less than one hundred and twenty (120) pounds of roofing cement or tar to the square of one hundred (100) feet. The entire surface then to be mopped over with roofing cement or tar and a cap sheet of wool felt applied. All joinings along walls and around openings to be carefully made. The roof to be then covered with a heavy coating of roofing cement or tar and screened gravel, not less than one (1) cubic yard of gravel to six hundred (600) square feet, gravel to be screened through ⅝-inch mesh and

free from sand and loam. All walls and openings to be flashed. All roofing cement and tar shall be applied hot.

In Building Construction and Superintendence, Part II, Kidder gives the following specifications for flashing a gravel roof:

"*Flashing.*—Finish the roofing against fire walls, chimneys, scuttle and skylight by turning the felt up 4 inches against the wall. Over this lay an 8-inch strip of felt with half its width on the roof. Fasten the upper edge of the strip and the several layers of felt to the wall by laths or wooden strips securely nailed, and press the strip of felt into the angle of the wall and cement to the roof with hot pitch. Nail the lower edge of the strip to the roof every 4 or 5 inches. Take special care in fitting around chimneys and skylights. Extend the felt up 6 inches on the pitch of the roof, and secure every 4 inches with 3d nails with tin washers."

The pitch should not be more than ¼ and should preferably be about ¾ to 1 inch to the foot. Gravel is sometimes used on roofs nearly flat.

Gravel roofing under ordinary conditions will last for from 10 to 15 years. With careful attention it can be made to last longer and has been known to last 30 years.

The cost of gravel roofing varies with local conditions and specifications. In various reports of the Association of Railway Superintendents of Bridges and Buildings costs of gravel roofs, not including the sheathing, per square are given as follows: Three-ply gravel roof in California, costs $3.75; four-ply (4) gravel roof in Kansas, costs $3.00; in Chicago, costs from $3.00 to $4.00; and in New England, costs from $4.00 to $5.00. The cost varies greatly with the specifications.

Prepared Gravel Roofing.—Prepared gravel roofings may be bought in the market. Prepared gravel roofing manufactured by the Armitage Manufacturing Company, Richmond, Va., was quoted at $2.50 per roll of 108 square feet and including nails and cement, delivered in central Illinois. This company has discontinued the manufacture of prepared slag roofing.

Slag Roofing.—Slag is sometimes used in the place of gravel in making roofs. The method of constructing the roof and the specifica-

tions are essentially the same as for a gravel roof. For detailed specifications for laying slag roofing see description of the Locomotive Erecting and Machine Shop, Philadelphia & Reading R. R., given in Part IV.

Asphalt Roofing.—Asphalt roofing is laid like tar and gravel roofing except that asphalt is used in the place of tar or cement. For discussion of the composition and properties of asphalt see Baker's Roads and Pavements, Chapter XIII. The following specifications will give a good roof:

Five (5) Ply Wool Felt, Trinidad Asphalt and Gravel Roof.—First cover the sheathing boards with one (1) thickness of dry felt, and over this put four (4) thicknesses of No. 1 wool roofing felt, weighing not less than fifteen (15) pounds (single thickness) to the square of one hundred (100) square feet. The felt to be smoothly and evening laid, and well cemented together the full width of the lap, not less than nine (9) inches between each layer, with Trinidad asphalt roofing cement, using not less than one hundred (100) pounds of asphalt to one square of one hundred (100) square feet. All joinings along the wall and around openings to be carefully made. The roof is then to be covered with a coating of asphalt and screened gravel, not less than one (1) cubic yard of gravel to six hundred (600) square feet of roof, gravel to be screened through a $\frac{5}{8}$-inch mesh and to be free from loam. All walls to be flashed with old style tin or galvanized iron, or a 2×4 is to be built into the walls to make roof connections to.

Five (5) Ply Combined Flax and Wool Felt, Trinidad Asphalt and Gravel Roof.—First cover the sheathing boards with one thickness of dry felt, over this put one (1) thickness of flax felt and three (3) thicknesses of No. 1 wool roofing felt, weighing not less than fifteen (15) pounds (single thickness) to the square of one hundred (100) square feet. The felt to be smoothly and evenly laid, and well cemented together the full width of the lap, not less than eleven (11) inches between each layer, with Trinidad asphalt roofing cement, using not less than one hundred (100) pounds of asphalt to the square of one hundred (100) square feet. All joinings along the walls and around openings to be carefully made. The roof is then to be covered with a coating of Trinidad asphalt roofing cement and screened gravel, not less than one (1) cubic yard of gravel to six hundred (600) square feet

of roof, gravel to be screened through a ⅝-inch mesh and to be free from loam. All walls to be flashed with old style tin or galvanized iron, or a 2 × 4 is to be built into the wall to make connections to.

Prepared asphalt roofing can be bought in the market. It is sold in rolls 36 inches wide and is laid in courses.

The Arrow Brand Ready Asphalt Roofing, manufactured by the Asphalt Ready Roofing Company, New York, was quoted in 1903 delivered in central Illinois as follows: Arrow Brand No. 1, sand surfaced, per roll $2.75; rolls contain 110 square feet which will cover 100 square feet of roof and weigh 80 pounds. Arrow Brand No. 2, gravel surfaced, per roll $2.75; rolls contain 110 square feet which will cover 100 square feet of roof and weigh 140 pounds. The necessary nails and asphalt required in laying the roofing are included in the above prices. This roofing is in use by a number of railways.

Shingle Roofs.—Shingle roofs are now very seldom used for mill buildings. Shingles have an average width of 4 inches and with 4 inches laid to the weather 900 are required to lay one square of roof. One thousand shingles require about 5 lbs. of nails. One man can lay from 1500 to 2000 shingles in a day of 8 hours. The cost of shingle roofs varies with the locality from, say, $3.25 to $6.25 per square.

Asbestos Roofing.—The "Standard" Asbestos Roofing, manufactured by the H. W. Johns-Manville Co., New York, is composed of a strong canvas foundation with asbestos felt on the under side, and satturated asbestos felt on the upper side finished with a sheet of plain asbestos; the whole being cemented together with a special cement and compressed together into a flexible roofing. It does not require painting, although it is commonly painted with a special paint, one gallon of which will cover about 150 square feet. The roofing is laid with a lap of 2 inches, beginning at the lower edge of the roof and running parallel to the eaves. The laps are cemented and are nailed with special roofing nails and caps. The roofing is laid on sheathing and is very easily and cheaply laid. It is quite flexible and may be used for flashing and for gutters. It is practically fireproof and makes a very satisfactory roof. Asbestos roofing comes in rolls and weighs about 75

pounds per square. It costs about $3.00 per square laid on the roof.

The above named company makes several other brands of asbestos roofing the cost of which is about the same as the "Standard."

Asbestos roofing felts may be purchased which are used for roofing in one, two or three-ply, and are laid in the same way as for gravel roofing.

Carey's Roofing.—Carey's Magnesia Flexible Cement Roofing, manufactured by the Philip Carey Manufacturing Company, Lockland, Ohio, is made by putting a layer of asphalt cement composition on a foundation of woolen felt and imbedding a strong burlap in the upper surface of the cement. After laying, the burlap is covered with a tough elastic paint which when it dries gives a surface similar to slate. The roof is practically acid proof and burns very slowly. It comes in rolls 29 inches wide and containing sufficient material to lay one square of roof. The roofing is made in two weights, standard weighing 90 pounds per square, and extra heavy weighing about 115 pounds per square. A special flap is provided on one side to cover the nail heads. The roofing is very pliable and can be used for flashing and for gutters. It should be laid on sheathing and is very easily and cheaply applied. It may be laid over an old shingle or corrugated iron roof. It costs about $2.75 to $3.25 per square laid on the roof.

Granite Roofing.—Granite Roofing, manufactured by the Eastern Granite Roofing Company, New York, is a ready-to-lay composition roofing with manufactured quartz pebbles imbedded in its upper surface. It is a very satisfactory roofing and is quite extensively used. It costs about $2.75 to $3.75 per square laid on the roof.

Ruberoid Roofing.—P. & B. Ruberoid Roofing, manufactured by the Standard Paint Co., New York, is quite extensively used and has given good satisfaction. The following description is taken from the maker's catalog: "No paper whatever is used in the manufacture of Ruberoid Roofing. It has a foundation of the best wool felt, except in the case of the ½-ply grade which is a combination of wool and hair. This is first saturated with the P. & B. water and acid proof compound,

and afterwards coated with a hard solution of the same material, thereby making the roofing at once light in weight as well as strong, durable and elastic. It is thoroughly acid and alkali proof, is not affected by coal gas or smoke and can be laid on either pitched or flat roofs, proving equally effectual in both cases. Inasmuch as it contains no tar or asphalt the roofing is not affected by extremes in temperature."

Ruberoid is made ½-ply weighing 22 pounds per square; 1-ply weighing 30 pounds per square; 2-ply weighing 43 pounds per square; and 3-ply weighing 51 pounds per square. The 2-ply and the 3-ply roofing are commonly used for factories and mills. The roofing is put up in rolls 36 inches wide, containing two squares (200 square feet), with an additional allowance of 16 square feet for two-inch laps at the seams; sufficient tacks, tin caps and cement are included with each roll.

Ruberoid roofing costs from $2.75 to $3.75 per square laid on the roof.

Ferroinclave.—This is a patented roofing made by the Brown Hoisting Machinery Co., Cleveland, Ohio, and is described in a letter to the author as follows: "Ferroinclave roofing is made by coating a special crimped or corrugated iron or steel on both sides with a mixture of Portland cement and sand, after which it is painted on the upper side. The sheets are made of No. 22 or No. 24 sheet steel, and full sized sheets are 20 inches wide and 10 feet long. The steel is crimped or corrugated with corrugations about 2 inches wide and ½ inch deep, the width of the corrugation on the outer side being less than on the inner side, thus forming a key to hold the cement mortar in place. The sheets are laid in the same manner as corrugated steel, and a coating of Portland cement mortar, composed of 1 part Portland cement and 2 parts sand, is plastered on the upper and lower surfaces to a thickness of ⅜ of an inch above and below the corrugations, making the total thickness of the roofing 1¼ inches. The weight of No. 24 sheet steel Ferroinclave is about 15 lbs. per square foot when filled with cement mortar as above. A test of a sheet of Ferroinclave made as above, showed failure with a uniformly distributed load of 300 lbs. per square foot with supports 4′ 10″ apart, the cement having set ten days. The

cost of this roofing is about $21.00 per square complete in place on the roof." The Brown Hoisting Machinery Co. has also used Ferroinclave quite extensively for floors and side walls of buildings.

Examples of Roofs.—The Boston Manufacturer's Mutual Fire Insurance Co., recommend the following roof for mill buildings: "Roofs of ordinary type may be only of plank covered with composition or other suitable roofing material. In special cases the roof should consist of a 3-inch plank, 1 inch of mortar, a 1-inch top board and a 5-ply composition roof. Such a roof is impervious to heat and cold."

The roof of the machine shop of the Chicago City Railway is composed of 2 × 6-in. tongued and grooved sheathing overlaid with 5 layers of "Cincinnati" wool felt, having 100 pounds of cement to 100 square feet, and is covered with tar and gravel.

The roof of the Lehigh Valley R. R. Shops at Sayre, Pa., is a slag roofing on armored concrete.

The roof of the Great Northern R. R. shops at St. Paul, Minn., has double sheathing with 1 × 3-in. strips between the layers to provide an air pace and prevent sweating. Monarch roofing is laid on the sheathing.

The roof of the Philadelphia & Reading shops, at Reading, Pa., is felt on plank sheathing covered with tar and slag.

The roof of the A. T. & S. F. R. R. machine shops at Topeka, Kas., is Ludowici tile laid on 2 × 2-in. timber strips.

The roof of the Union Train Shed at Peoria, Ill., is Ludowici tile laid on angle sub-purlins as shown in Fig. 87.

Roof Coverings for Railway Buildings.—The following abstract of the report of the committee on roof coverings presented at the annual meeting of the Association of Railway Superintendents of Bridges and Buildings, 1902, will give a very good idea of the present practice in covering railroad buildings.

"Slate is much used for station buildings where there is not much climbing for repair of skylights or telegraph wires. It has a life of from 35 to 40 years, and the roof should have a pitch of not less than 6 inches per foot. Vitrified tile is very durable where rightly made and laid on steep roofs, but is not adapted for ordinary railroad buildings. Shingle roofs last as long as 28 years, and should be laid with

6 inches pitch per foot; they are very satisfactory where slate is too expensive. For flat roofs a tar and gravel composition is preferred and will last 12 to 18, and even 20 years. Slag or broken stone of the size of peas is sometimes used in the place of gravel. In such roofs, much depends upon the paper used, the pitch, and the thoroughness of the work; 3-ply is too light, 4-ply is good, but 5-ply is better. Asphalt pitch is sometimes preferred to coal-tar, but the latter is sufficiently durable. An asphalt-gravel roof must slope not more than ½ inch to the foot, on account of the liability to run in hot weather; but tar-gravel roofs may have a pitch of 1 inch per foot. With such very flat roofs as are required for asphalt, any settlement will form hollows that will hold water.

"Sheet metal roofs, corrugated or flat are not durable. Steel is less durable than iron and will last only about one year, where exposed to engine gases. Tin shingles of good quality give good results. Painted shingles have a short life unless frequently painted.

"Of patented roof coverings, Sparham is pulverized talcose lime rock, mixed with coal-tar pitch and applied hot to the roof with a trowel. This may be used for a flat roof or for a roof with a pitch of 3 or 4 inches per foot. Ruberoid is a wool felt saturated with a parafine preparation. Perfected Granite Roofing is 2-ply tarred paper with sea grit on one side. Both of these last may be used on any roof with a pitch of not less than 2 inches per foot. Cheap roofs made from roofing papers require mopping with tar, and if thus treated every two years (before the paper is bare) will last almost indefinitely. In railway work, however, roofs are generally left without attention until leaks are reported, when it is too late for mopping to do any good.

"Roofs requiring treatment every two years can hardly be considered as permanent. Slate for pitched roofs and tar and gravel for flat roofs are as nearly permanent as can be obtained for railway buildings.

"The cost per square of 100 square feet for roofs of different kinds in New England is as follows:

Slate	$ 9.00 to $12.00	Tar and gravel	$4.00 to $5.00
Tile	30.00 to 33.00	Sparham	5.00 to 5.50
Shingles		Ruberoid	2.75 to 3.75
Sawed cedar	4.50 to 5.00	Prefected Granite	2.75 to 3.25
Tinned	5.00 to 6.50	Paroid	3.00 to 3.50
Sheet tin, standing		2-ply double	2.00 to 2.25
seam	6.50 to 8.00	3-ply single	1.50 to 2.00"

CHAPTER XX.

Side Walls and Concrete Buildings.

SIDE WALLS.—The sides of steel frame mill buildings are covered with corrugated steel, expanded metal and plaster, wire lath and plaster, or with Ferroinclave a patent covering made of special corrugated steel and plaster, manufactured by the Brown Hoisting Co., Cleveland, Ohio.

Corrugated Steel.—The methods of fastening corrugated steel to the sides of buildings are the same as on the roof and are described in detail in Chapter XVIII. Where warmth is desired, buildings covered with corrugated steel are often lined with No. 26 corrugated steel with 1¼-inch corrugations. Where this lining is used spiking pieces should be bolted to the girts and intermediate spiking pieces should be placed between the girts to which to nail the lining. If this is not done the corrugated steel will gape open for the reason that it is impossible to rivet the side laps of the lining. Where anti-condensation lining is used on the sides it is made the same as on the roof except that the girts should always be placed not more than one-half the usual distance. The clinch-nail fastening is the best method for fastening the corrugated steel where the anti-condensation lining is used.

Expanded Metal and Plaster.—The methods of making walls of expanded metal and plaster are shown in Fig. 131 and Fig. 132, which show details of the construction of the soap factory buildings of W. H. Walker, Pittsburg, Pa. These buildings were constructed as follows: The buildings were made with a self-supporting steel frame, all connections except those for the purlins and girts being riveted. Inaccessible surfaces were painted with red lead and linseed oil before erection and the entire framework was painted two coats of graphite paint after

erection. The trusses are spaced from 14 to 18 feet and carry 6, 7 and 8-inch channel purlins. The purlins are spaced from 6 to 7 feet apart and carry roof slabs 2½ to 3 inches thick made of expanded metal and concrete. The expanded metal is made from No. 16 B. W. G. steel plate with 4-inch mesh, and the concrete is composed of 1 part Portland cement, 2 parts sand and 4 parts screened furnace cinders. The roof slabs are covered with 10 × 12-inch slate nailed directly to the

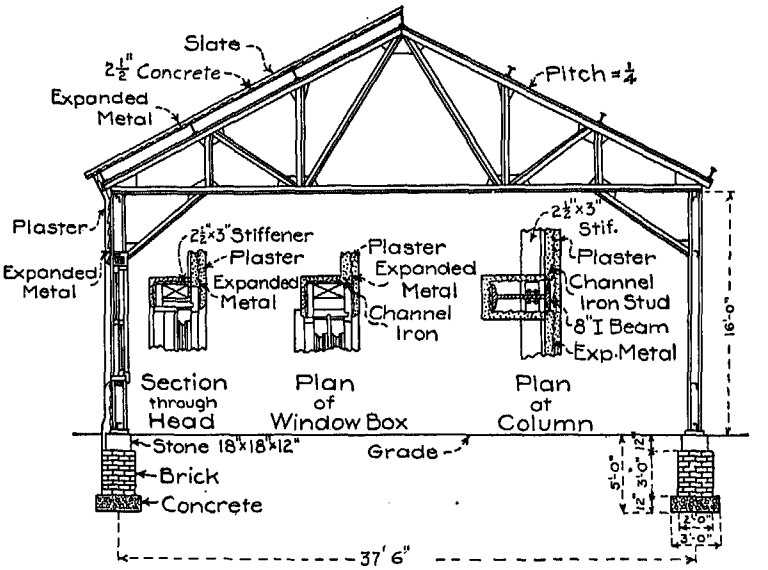


FIG. 131. CROSS-SECTION OF STEEL BUILDING COVERED WITH EXPANDED METAL AND PLASTER.

concrete, and are plastered smooth on the under side. The side walls were made by fastening ¾-inch channels at 12-inch centers to the steel framing, and covering this framework with expanded metal wired on. The expanded metal was then covered on the outside with a coating of cement mortar composed of 1 part Portland cement and 2 parts sand and on the inside with a gypsum plaster, making a wall about 2 inches thick. The ground floors were made by covering the surface with a 6-inch layer of cinders in which were imbedded 2 × 4-in. white pine nail-

ing strips 16 inches apart, and on these strips was laid a floor of tongued and grooved maple boards 1¼ inches thick and 2½ inches wide. The upper floors are made of concrete slabs reinforced with expanded metal, and supported on beams spaced 4 to 15 feet apart. Where the spans exceed 7 feet suspension bars 7″ × ⅜″ were placed 3 feet apart and were bent around the flanges of the beams. The concrete filling was composed of 1 part Portland cement, 2 parts sand and 6 parts cinders. (For another description of this building see Engineering Record, August 25, 1900.)

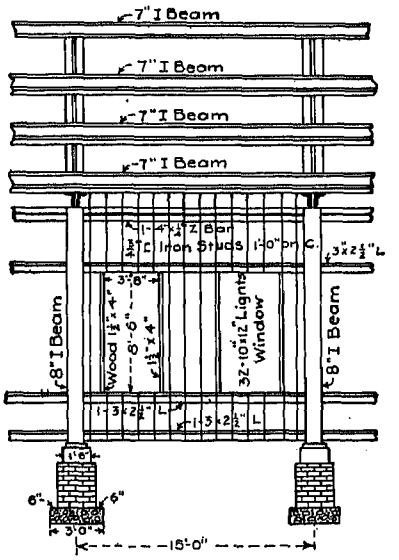


FIG. 132. SIDE ELEVATION OF STEEL BUILDING COVERED WITH EXPANDED METAL AND PLASTER.

The Northwestern Expanded Metal Co. now recommends that the first coat of the plaster used for curtain walls be composed of two parts lime paste, 1 part Portland cement and 3 parts sand, and that the wall be finished with a smooth coat composed of 1 part Portland cement and 2 parts sand.

For coating on wire lath the following has been found to give satisfactory results in Chicago and vicinity: For the first coat use a

mortar composed of 1 part Portland cement and 2 parts ordinary lime mortar. The lime should be very thoroughly slaked before using as the presence of any free lime will injure the wall. After the first coat has hardened it is thoroughly soaked and a finishing coat composed of 1 part Portland cement, 2 parts sand and a small quantity of slaked lime is applied and rubbed smooth.

A method of plastering curtain walls is described by Mr. George Hill in the Transactions of the American Society of Civil Engineers, Vol. 29, as follows: "The external curtain walls were composed of hard plaster, Portland cement and sand in equal parts, the scratch coat being applied to uncoated metallic lath, making the thickness of the scratch coat about 1 inch; then a surfacing of Portland cement ½-inch thick was applied on each side making the curtain walls a total thickness of 2 inches. Good results were obtained in every case except one, where the scratch coat was alternately frozen and thawed several times, and the outer surfacing of the wall peeled off in patches."

The Northwestern Expanded Metal Co. does not recommend the use of hard or patent plasters for curtain walls.

Expanded metal and plaster curtain walls are light, strong and efficient. They do not require the heavy foundations required by brick and stone walls and are fireproof. They can be used to advantage where it is desirable to have a large glass area in the sides of buildings. This type of construction is almost ideal for factory construction and will be much used in the future. There are quite a number of different systems but the methods of construction are essentially the same in all.

Curtain walls are made of wire lath and plaster in the same way as expanded metal and plaster and have all the advantages of the latter.

The cost of curtain walls made as described above is about $1.50 to $1.80 per square yard.

For a detailed description of the construction of small cement and steel buildings see Engineering Record, March 26, 1898.

Concrete Slabs.—The construction of reinforced concrete slabs patented by Milliken Brothers, New York, is described in Engineering

Record, December 22, 1900, as follows: "The slabs used on the roof of the concrete stable built for the Anglo-Swiss Condensed Milk Company, Brooklyn, N. Y., were 4 feet wide and about 15 feet long and were constructed as follows: Each slab has a steel frame with three 2 × ¼-inch transverse strips set edgewise at the ends and middle, and connected by longitudinal ¼-inch rods about 3⅝ inches apart so as to form a gridiron. The rods are set in staggered holes in the edge of the bars and form a framework over and under which No. 14 transverse wires are woven 6 inches apart. The lower surface of the frame is covered with open mesh fine-wire netting, wired around the edges, and the frame is filled with 1 : 2 : 4 Portland cement concrete made with very fine broken stone. The slab is 2 inches thick and has offset edges to make scarfed joints which are set with cement mortar. Voids are left in the concrete at the edges of the slabs to permit thin flat steel bars or angle clips to be bolted to the frames, and to be bolted to or locked around the framework. Then the holes are flushed with cement mortar and a ¼-inch surface coat is plastered over the slabs for the final finish. These slabs have been used for side and partition walls as well as for roof sheathing."

Masonry Walls.—Walls for filling in between the columns of mill buildings are commonly made very light, being usually determined by the clearance and the height. For buildings with 20 to 25 ft. posts, 8-inch walls are very commonly used. Where the columns are placed inside of the line of the walls, a greater thickness of wall is used than above; 13 and 17-inch walls being quite common.

The thickness of factory and warehouse walls which support roof trusses is about as given in Table XXII.

The thickness of the wall may be decreased when pilasters are used to assist in supporting the trusses.

For detailed information on the construction of brick and stone walls see Baker's Masonry Construction and Kidder's Building Construction and Superintendence, Part I.

TABLE XXII.

THICKNESS OF WAREHOUSE WALLS.

Height of Wall. Feet.	Thickness of Wall.	
	Brick. Inches.	Stone. Inches.
25	16	20
50	20	24
75	24	36

CONCRETE BUILDINGS.—Within the last few years quite a number of factory buildings have been constructed of concrete. Most of these buildings are monolithic, although recently quite a number of patents have been issued for concrete building blocks. The walls are usually made hollow when made monolithic or made of concrete blocks; the air space prevents the passage of dampness through the walls, makes the building warmer and is less expensive than to make the wall solid. In monolithic concrete construction the roof, floors and the angles in the walls are reinforced with metal put in according to some one of the many systems now in use.

The following abstract of the description of the construction of a monolithic concrete building, printed in the Engineering Record, July 30th, and August 20th., 1898, will give the reader an idea of the methods employed.

"The factory of the Pacific Coast Borax Company, at Constable Hook, Bayonne, N. J., is about 200 × 250 feet in extreme dimensions, and is partly one story and partly four stories in height. All the floors, floor beams, walls, columns, etc. are constructed of reinforced concrete on the Ransome system, built in molds so as to form a monolithic structure continuous throughout, except for the shrinkage joints dividing it into separate panels.

"The columns are supported on concrete footings reinforced with twisted steel bars. The walls of the building were built solid at the ends of the floor beams and the intermediate portions were made hol-

low by inserting wooden fillers, which were afterwards removed. The walls of the four story portion are 16 inches in extreme thickness up to the third floor, and are 15 inches above that point. The hollow walls have 3 to 4 inches of concrete on each side of the air space. Both the walls and the columns were bonded by vertical bars of twisted steel ¾-inch square, extending through them continuously from top to bottom, and similar rods were carried through the buildings from side to side transverse to the beams imbedded in the different floors about 12 feet apart so as to provide a certain transmission of the strain across the building and assure the resistance of the structure as a whole under the action of eccentric loads and pressures.

"At about every 25 feet in the length of the walls a vertical space of ¼ of an inch was made, extending from top to bottom and separating the wall into distinct sections. At each of these joints a twisted ¾-inch rod was imbedded from top to bottom on each side of the space. Similar rods were also placed at the corners of the building. Where the vertical shrinkage joints occur in the outside walls the continuity of the structure is preserved by carrying through them horizontal longitudinal pieces of twisted ¾-inch square rods about 2 feet long and set about 2 feet apart throughout the height of the wall.

"The columns were built in 16-foot sections, each section being one story in height, and were constructed by ramming the concrete inside of forms. The vertical boards composing these forms were made in short lengths, breaking joints over the cross pieces, and were placed in position as the concrete was placed in position. The forms were allowed to stand until required for another story, often remaining in position for several weeks, although it was considered that the concrete was strong enough to permit their removal when 48 hours old. The walls were laid up between vertical surfaces of plain 1½-inch plank, laid horizontally and secured by tie bolts running through the molds. The wall was built up in sections about four feet in height and the concrete was laid in continuous 6-inch layers, extending entirely around the circumference of the wall, and was thoroughly rammed as deposited. After the concrete had set sufficiently, the bolts were loosened and the boards forming the sides of the mold were pulled up and set in position for building another zone of wall. About 35 men were at work building the walls and constructed an average of 2000 square feet a day.

"The partitions in the building are 2 inches thick and are made of solid concrete reinforced by a framework of twisted ¼-inch bars about 2 feet apart, both vertically and horizontally. The partitions were built in molds, and set so that the face comes exactly even with the edge of a shrinkage joint in the floor, and always set over a floor beam.

"The concrete was made of Atlas cement and broken basaltic rock, all of which will pass through a 2-inch ring and most of which will pass through a 1-inch ring. The unscreened rock was mixed with cement in the following proportions: For foundations, 1 to 10; for the walls, floors and most of the work, 1 to 6½; for the columns, 1 to 5; and for the lower chords of the floor beams, 1 to 6, using very fine stone."

In constructing the Ingalls office building in Cincinnati, Ohio,—described in Engineering News, July 30, 1903, and Engineering Record, July 18, 1903—the methods used were essentially the same as described above with a few exceptions which will be noted.

"The broken stone included the total product of the crusher and was fine enough to pass through a 1-inch screen. The concrete was mixed rather wet to insure complete filling of all interstices around the bars. Enough water was used to always give a semi-fluid consistency which allowed puddling rather than ramming. It was made wetter for the columns than for the floors and girders because the bars interferred with the ramming in the molds for the columns. The columns were built in one-story lengths and the concrete was rammed in the molds in layers not more than 12 inches deep. The concrete was dumped from the floor above into the bottom of the mold. The steel rods were placed in position before the concreting was commenced, and were wired in position. A force of 28 men working with hoisting machines and wheelbarrows placed about 100 cubic yards of concrete in a day."

The present tendency in concrete building construction is toward the use of a concrete made of Portland cement and finely crushed stone, mixed very wet and deposited in the molds practically without ramming. The concrete must be rich in cement to make a good wall under these conditions.

Surface Finish.—Where it is desired to imitate stonework, imitation joints are formed in the face of the wall and the surface is either picked while the concrete is yet tender or is tooled after the concrete has hardened. Bush hammering of concrete walls can be done by an ordinary workman for from 1½ to 2 cts. per sq. ft. Where the concrete is coarse a coating of cement mortar may be applied as the concrete is placed in the molds by means of a piece of sheet steel placed from 1 to 2 inches from the forms; the cement mortar, usually made of 1 part Portland cement and 2 parts sand, is then rammed into the vacant space, after the main body of the concrete has been rammed in place, and the piece of sheet steel is removed.

The preparation of the forms requires considerable study to obtain a smooth surface and unbroken corners. The use of matched or tongued-and-grooved stuff is not desirable as the concrete fills the openings made by shrinkage and there is no room to expand. Unmatched boards dry apart and let the water in the concrete leak out, carrying with it some of the cement. The best way to build the forms is to use narrow stuff and bevel one edge of the boards; the sharp edge of the bevel lying against the square edge of the adjoining board allows the edge to crush when swelling and closes up the joint. A coat of soft soap applied to the forms before filling, prevents the concrete from adhering. The soap should be scraped and brushed off with a steel brush as the forms are removed.

For description of concrete round house at Moose Jaw, Canada, see Part IV.

CHAPTER XXI.

Foundations.

Introduction.—The design of the foundations for mill buildings is ordinarily a simple matter for the reason that the buildings are usually located on solid ground and the loads on the columns are small. Where the soil is treacherous or when an attempt is made to fix the columns at the base the problem may, however, become quite complicated.

Bearing Power of Soils.—The bearing power of a soil depends upon the character of the soil, its freedom from water, and its lateral support. The downward pressure of the surrounding soil prevents lateral displacement of the material under the foundation and adds materially to the bearing power of treacherous soils.

The safe bearing power of soils given in Table XXIII may be used as an aid to the judgment in determining on a safe load for a foundation. However no important foundations should be built without making careful soundings and bearing tests.

A soil incapable of supporting the required loads may have its supporting power increased (1) by increasing the depth of the foundation; (2) by draining the site; (3) by compacting the soil; (4) by adding a layer of sand or gravel; (5) by using timber grillage to increase the bearing area; (6) by driving piles through the soft stratum, or far enough into it to support the loads.

A method used in France for compacting foundations is to drive holes with a heavy metal plunger and then fill these holes with closely rammed sand or gravel.

Several kinds of patented concrete piles are now in use to a limited extent in this country for building foundations.

TABLE XXIII.

SAFE BEARING POWER OF SOILS.*

Kind of Material.	Safe Bearing Power in Tons per Sq. Ft.	
	Min.	Max.
Rock-hardest in thick layers in bed....	200	
" equal to best ashlar masonry....	25	30
" " " " brick.....	15	20
" " poor brick.....	5	10
Clay in thick beds, always dry.....	4	6
" " " " moderately dry.....	2	4
" soft.....	1	2
Gravel and coarse sand, well cemented	8	10
Sand compact and well cemented.....	4	6
" clean, dry.....	2	4
Quicksand, alluvial soils, etc.....	0.5	1

When foundations are placed on solid rock, the surface of the rock should be carefully cleaned of loose and rotten rock and roughly brought to a surface as nearly perpendicular to the direction of the pressure as practicable. A layer of cement mortar placed directly on the rock surface will assist in bonding the foundations and the footing together.

When foundations are placed on sand, gravel or clay it is usually only necessary to dig a trench and start the foundation below frost. If the soil is somewhat yielding or if the load is heavy the foundation should be carried to a greater depth or the footings should be made wider than for greater depths.

Bearing Power of Piles.—Probably no subject has been more freely discussed and with more conflicting views and opinions than has the safe bearing power of piles. The safe load to put on a pile in any particular case is dependent upon so many conditions that any formula for the safe bearing power is necessarily simply an aid to the judgment of the engineer, and not an infallible rule to be blindly followed. All

*Treatise on Masonry Construction, by Ira O. Baker,—John Wiley & Sons, Publishers, New York.

formulas for the bearing power of piles determine the safe bearing power from the weight of the hammer, the length of free fall of the hammer, and the penetration of the pile. The penetration of the pile for any blow of the hammer depends on the condition of the head of the pile, upon whether the pile is driving straight, and upon the rigidity of the pile. The penetration of a slim, limber pile with a broomed head is very misleading, and any formula will give values too large.

The Engineering News formula for the safe bearing power of piles is most used and is certainly the most reliable. It is

$$P = \frac{2\,W\,h}{s+1}$$

where P = safe load on pile in tons;
W = weight of hammer in tons;
h = distance of free fall of the hammer in feet;
s = penetration of the pile for the last blow in inches.

If the pile is driven with a steam hammer the factor unity in the denominator is changed to one-tenth. This formula is supposed to give a factor of safety of about 6, and has been shown by actual use to give values that are safe.

Where piles are to be driven through gravel or very hard ground the lower ends are often protected with cast iron or steel points. The value of these points is questionable and most engineers now prefer to drive piles without their use, simply making a very blunt point on the pile. In driving piles, care must be used where small penetrations are obtained not to smash or shiver the pile. Piles driven to a good refusal with a penetration of, say, 1 inch for the last blow, with a fall of 20 ft. and a 2000-lb. hammer will safely support almost any load that can be put on them.

Piles are usually driven at about 3-ft. centers over the bottom of the foundation. After the piles are driven they are sawed off below the water level and (1) concrete is deposited around the heads of the piles, or (2) a grillage or platform is built on top of the piles to support the walls

or piers. The first method is now the most common one for mill building foundations.

Pressure of Walls on Foundations.—In Fig. 133, let W = resultant weight of the wall, the footing and the load on the wall, l = length of the footing and b = distance from center of gravity of footing to point of application of load W, and let the wall be of unit

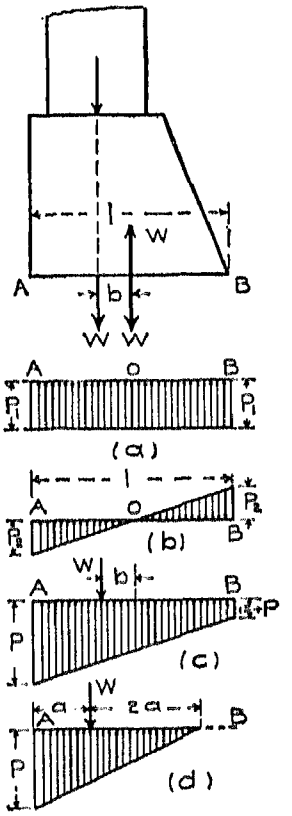


FIG. 133.

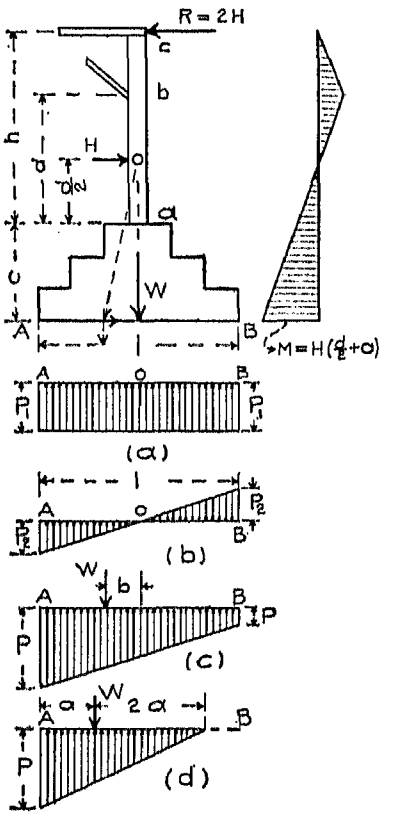


FIG. 134.

length. The pressure on the footing will be that due to direct load W, and a couple with an arm b and a moment = + Wb. The pressure due to the direct W will be

$$P_1 = W \div l$$ as shown in (a),

and the maximum pressure due to the bending moment, $M = + Wb$, will be

$$P_2 = \frac{Mc}{I} = \frac{6\ W\ b}{l^2}$$

The pressure at A will be

$$P = P_1 + P_2 = \frac{W}{l} + \frac{6\ W\ b}{l^2} \qquad (83)$$

and at B will be

$$P = P_1 - P_2 = \frac{W}{l} - \frac{6\ W\ b}{l^2} \qquad (84)$$

as shown in (c).

Now if P_1 is made equal to P_2 the pressure at B will be zero and at A will be twice the average pressure. Placing $P_1 = P_2$ in (84) and solving for b, we have $b = \frac{1}{6}\ l$. This leads to the theory of the middle third or kern of a section. If the point of application of the load never falls outside of the middle third there will be no tension in the masonry or between the masonry and foundation, and the maximum compression will never be more than twice the average shown in (a).

If the point of application of the load falls outside the middle third (b greater than $\frac{1}{6}\ l$) there will be tension at B, and the compression at A will be more than twice the average. But since neither the masonry nor foundation can take tension, formulas (83) and (84) will give erroneous results.

In (d) Fig. 133, assume that b is greater than $\frac{1}{6}\ l$, and then as above, the load W will pass through the center of pressures which will vary from zero at the right to P at A. If 3 a is the length of the foundation which is under pressure, then from the fundamental condition for equilibrium for translation, summation vertical forces equals zero, we will have

$$W = \tfrac{1}{2}\ P\ 3\ a \qquad \text{and}$$

$$P = \frac{2\ W}{3a} \qquad (85)$$

Pressure of a Pier on Foundation.—In Fig. 134, let W = resultant of the stresses in the column and the weight of the pier, l = length, c = depth and n = the breadth of the footing of the pier in feet. The bending moment at the top of the pier is $M = -\frac{1}{2} H d$ and at the base of the pier is $M_1 = -H(\frac{1}{2} d + c)$. Now the pier must be designed so the maximum pressure on the foundation due to W and the bending moment M_1 will not exceed the allowable pressure. The maximum pressure on the foundation will be

$$P = P_1 \pm P_2 = \frac{W}{ln} \pm \frac{M_1 y}{I}$$

$$= \frac{W}{ln} \pm \frac{H\left(\frac{d}{2} + c\right)\frac{l}{2}}{\frac{1}{12} n l^3}$$

$$= \frac{W}{ln} \pm \frac{3 H (d + 2c)}{nl^2} \qquad (86)$$

It will be seen from (86) that a shallow pier with a long base is most economical.

To find the relations between l and c when the maximum pressure is twice the average, place

$$\frac{W}{ln} = \frac{3 H (d + 2c)}{n l^2}$$

and

$$l = \frac{3 H (d + 2c)}{W} \qquad (87)$$

For any given conditions the value of l that will be a minimum may be found by substituting in the second member of (87).

To illustrate the method of calculating the size of a pier we will calculate the pier required to fix the leeward column in Fig. 57.

The sum of the stresses in column A-17 is a minimum for dead and wind load and will be (Table V) equal to 4800 + 4500 = 9300 lbs.

Try a pier 3′ 0″ × 3′ 0″ on top, 6′ 0″ × 6′ 0″ on the base and 6 feet deep, weighing about 16,700 lbs.

Substituting in (86) we have

$$P = \frac{26{,}000}{36} \pm \frac{3 \times 4300 (14 + 12)}{6 \times 36}$$

$$= 722 \pm 1553$$

This gives tension on the windward side which will not do, and so we will reinforce the footing with beams and make $l = 10$ ft., and increasing weight so that $W = 40800$ lbs.

$$P = 680 \pm 559$$
$$= 1239 \text{ or } 121 \text{ lbs. per square foot, which}$$

is safe for ordinary soils.

If it had been necessary to drive piles for this pier, a small amount of tension might have been allowed on the windward side if the tops of the piles had been enclosed in concrete.

Design of Footings.—The thickness and length of the offsets in a concrete or masonry footing are commonly calculated as for a beam fixed at one end and loaded with a uniform load over the projecting part equal to the maximum pressure on the footing. If p = projection of the footing in inches; t = the thickness of the footing in inches; P = pressure on foundation in pounds per sq. ft.; and S = safe working load of the material of which the footing is made in pounds per square inch, by substituting in the fundamental formula for flexure and solving for p,

$$p = 4t\sqrt{\frac{3S}{P}} \qquad (88)$$

The values of S in common use are: first class Portland cement concrete 50 lbs.; ordinary concrete 30 lbs.; limestone 150 lbs.; granite 180 lbs.; brickwork in cement 50 lbs.

The projection and thickness of the footing course is sometimes calculated on the assumption that the footing course is a beam fixed at the center, in place of as above. This solution hardly appears to be justified.

Pressure of Column on Masonry.—The following pressures in pounds per square inch are allowed by the building laws of New York.—Portland cement concrete 230 lbs.; Rosendale cement concrete 125 lbs.; Rubble stonework laid in Portland cement mortar 140 lbs.; brickwork laid in Portland cement mortar 250 lbs.; brickwork laid in lime mortar 110 lbs.; granite 1000 lbs.; limestone 700 lbs. It is very common to specify 250 lbs. per square inch for bearing on good Portland cement pedestals, and 300 lbs. per square inch is not uncommon.

Allowable Pressures.—The following unit pressures have been proposed by Mr. C. C. Schneider in "Structural Design of Buildings." *

1. *Foundations.*—Pressure on foundations not to exceed, in tons per square foot:

Soft clay	1
Ordinary clay and dry sand mixed with clay	2
Dry sand and dry clay	3
Hard clay and firm, coarse sand	4
Firm, coarse sand and gravel	6

2. *Masonry.*—Working pressure in masonry not to exceed, in tons per square foot:

Common brick, Rosendale-cement mortar	10
" " Portland-cement mortar	12
Hard-burned brick, Portland-cement mortar	15
Rubble masonry, Rosendale-cement mortar	8
" " Portland-cement mortar	10
Coursed rubble, Portland-cement mortar	12
First-class masonry, sandstone	20
" " " limestone	25
" " " granite	30
Concrete for walls:	
Portland cement 1-2-5	20
" " 1-2½4	25

3. *Pressure on Wall-Plates.*—The pressure of beams, girders, wall plates, column bases, etc., on masonry shall not exceed the following, in pounds per square inch.

On brickwork with cement mortar	200
" rubble masonry with cement mortar	200
" Portland-cement concrete	350
" first-class sandstone	400
" " " limestone	500
" " " granite	600

4. *Bearing Power of Piles.*—The maximum load carried by any pile shall not exceed 40,000 lbs., or 600 lbs. per sq. in. of its average cross-sections.

* Trans. Am. Soc. C. E., Vol. 54, 1905.

Piles driven in firm soil to rock may be loaded to the above limits. Piles driven through loose, wet soil to solid rock or equivalent bearing shall be figured as columns with a maximum unit strain of 600 lbs. per sq. in., properly reduced.

The minimum distance between centers of piles shall be 2½ ft.

5. *Loads on Foundations.*—The live loads on foundations shall be assumed to be the same as for the footings of columns. The areas of the bases of the foundations shall be proportioned for the dead load only. That foundation which receives the largest ratio of live to dead load shall be selected and proportioned for the combined dead and live loads. The dead load on this foundation shall be divided by the area thus found, and this reduced pressure per square foot shall be the permissible working pressure to be used for the dead load of all foundations.

6. *Reduction of Live Load on Columns.*—For columns carrying more than five floors, these live loads may be reduced as follows:

For columns supporting the roof and top floor, no reductions;

For columns supporting each succeeding floor, a reduction of 5 per cent of the total live load may be made until 50 per cent is reached, which reduced load shall be used for the columns supporting all remaining floors.

CHAPTER XXII.

Floors.

Introduction.—The requirements and the local conditions governing the design of floors for shops and mills are so varied and diversified that the subject of floor design can be treated only in a general way. Floors will be discussed under the head of (1) ground floors and (2) floors above ground.

GROUND FLOORS.—Types of Floors.—There are three general types of ground floors in use in mills and shops: (1) solid heat conducting floors as stone, brick or concrete; (2) semi-elastic, semi-heat conducting floors as earth, macadam or asphalt; (3) elastic non-heat conducting floors of wood or with a wooden wearing surface.

(1) Floors of the first class have been used in Europe and formerly in this country to quite an extent in shops and mills, and at present are much used in round houses, smelters, foundries and in other buildings where the wear and tear are considerable or where men are not required to stand alongside a machine. Floors of this class are cold and damp and make workmen uncomfortable. The wooden shoes of the continental workmen or the wooden platforms in use in many of our shops which have floors of this class, overcome the above objections to some extent. The gritty dust arising from most concrete floors is very objectionable where delicate machinery is used. The noise and danger from breakage and first cost are additional objections to floors of this class.

(2) Floors of this class have many of the objections and defects of floors of the first class. These floors are liable to be cold and damp unless properly drained, and give rise to a gritty dust that is often intolerable in a machine shop.

Earth and cinder floors are very cheap and are adapted to forge shops and many other places where concrete and brick floors are now put down. Floors of this class should be well tamped in layers and should be carefully drained. Tar-concrete and asphalt floors are more elastic and conduct less heat than any of the floors above mentioned, but the surface is not sufficiently stable to support machinery directly, and floors of this class are very much improved by the addition of a continuous wooden wearing surface.

(3) Floors of wood or with a wooden wearing surface appear to be the most desirable for shops, mills and factories. Wooden floors are elastic, non-heat conducting and are pleasant to work on. They are cheap, easily laid, repaired and renewed. They are easily kept clean and do not give rise to grit and dust.

The most satisfactory wearing surface on a wooden floor is rock maple ⅞ to 1¼ inches thick and 2½ to 4 inches wide, matched or not as desired. The matched flooring makes a somewhat smoother floor and is on the whole the most satisfactory. The wearing floor should be laid to break joints and should be nailed to planking or stringers laid at right angles to the surface layer. The thickness of the planking will depend upon the foundation and upon the use to which the floor is to be put.

The different classes of floors will now be briefly discussed and illustrated by examples of floors in use.

Cement Floors.—The construction of cement or concrete floors is similar to the construction of cement sidewalks, the only difference being that the floor usually requires the better foundation. The foundation will depend upon the use to which the floor is to be put, and upon the character of the material upon which the foundation is to rest. The excavation should be made to solid ground or until there is depth enough to allow a sub-foundation of gravel or cinders. Upon this base a layer of cinders or gravel 6 to 8 inches thick is placed and thoroughly rammed. The cement concrete base, made of 1 part Portland cement, 3 parts sand and 5 to 6 parts broken stone or gravel, is then placed on

the sub-foundation and thoroughly rammed. The cement and sand should be mixed dry until the mixture is of a uniform color, the gravel or broken stone is then added, having previously been wet down, and the concrete is thoroughly mixed, sufficient water being added during the process of mixing to make a moderately wet concrete. The concrete is of the proper consistency if the moisture will just flush to the top when the concrete is thoroughly rammed. The concrete should be mixed until the ingredients are thoroughly incorporated and each particle of the aggregate is thoroughly coated with mortar.

The wearing coat is usually made of 1 part Portland cement and one or two parts of clean sharp sand or granite screenings that will pass through a ¼-inch screen. The thickness of the wearing coat will depend upon the wear, and varies from ¼ to 2 inches thick, 1 inch being a very common thickness. The mortar for the wearing surface should be rather dry and should be applied before the cement in the concrete base has begun to set. Care should be used to see that there is not a layer of water on the upper surface of the base or that a film of clay washed out of the sand or gravel has not been deposited on the surface, for either will make a line of separation between the base and the wearing surface. The mortar is brought to a uniform surface with a straight edge, and is rubbed and compressed with a float to expel the water and air bubbles. As the cement sets it is rubbed smooth with a plastering trowel. Joints should be formed in the floor making it into blocks about 4 to 8 feet square.

Cement floors are said to be a failure for railway round houses for the reason that they flake and crack after they have been used a short time, on account of the varying changes to which they are subjected.

Cement floors vary in cost, depending upon the thickness of the floor and upon local conditions. In central Illinois a cement floor having a 1-inch surface coat and 3 inches of concrete laid on a cinder foundation 6 to 8 inches thick can be obtained (1903) for about 12 cents per square foot. A very substantial concrete floor can usually be obtained for about 20 cents per square foot.

Tar Concrete Floors.—The following specifications for tar concrete floors are given in circulars Nos. 54 and 55 of the Boston Manufacturer's Mutual Fire Insurance Co., and are reprinted in Engineering News, March 21, 1895.

"The floor to be 6 inches thick, and to be put down as follows: The lower 5 inches to be of clean coarse gravel or broken stone, with sufficient fine gravel to nearly fill the voids, thoroughly coated with coal-tar and well rammed into place. On this place a layer 1 inch thick of clean, fine gravel and sand heated and thoroughly coated with a mixture of coal-tar and coal-tar pitch in the proportions of 1 part of pitch and 2 parts of tar. This layer is to be rolled with a heavy roller and brought to a true and level surface ready to receive the floor plank. No sand or gravel to be used while wet.

"A floor of the kind above specified should always be protected by a floor of wood over it, and the plank should be laid and bedded in the top surface while it is warm and before it becomes hard.

"For light work the thickness of the lower layer of concrete may be reduced one or more inches if upon a dry gravelly or sandy soil. For storage purposes where the articles stored are light and trucks are little used, the following specification has been found to give a satisfactory floor:

"The lower layer being mixed and put down as above specified, the top layer will be of fine gravel and sand, heated and thoroughly mixed with a mixture of equal parts of coal-tar, coal-tar pitch and paving cement, so that each particle of sand and gravel is completely coated with the mixture, using not less than one gallon of the mixture to each cubic foot of sand and gravel. This layer should be well rolled with a heavy roller and allowed to harden several days before being used."

Brick Floors.—Brick floors are recommended as the most satisfactory floors for round houses. Round house floors on the Boston & Maine R. R. are made as follows: *Brick is laid flat on a 2-inch layer of bedding sand on well compacted earth, gravel or cinders. Joints are left open ⅜ of an inch and are swept full of cement grout.

Round house floors are made on the Chicago, Milwaukee & St. Paul R. R., as follows: *Vitrified brick is laid on edge on a layer of sand 1

*Eighth Annual Report of the Association of Railway Superintendents of Bridges and Buildings.

to 2 inches thick on a cinder foundation 6 inches thick. Fine sand is broomed into the cracks after the brick are in place.

The cost of this floor per square yard is about as follows:

Material.

Firebox cinders cost nothing	$00.00
Paving brick	0.50

Labor.

Preparing the foundation	0.20
Laying the brick	0.15
Total cost per square yard	$0.85
Total cost per square foot	9½ cents.

The cost of brick floors as given in the reports of the Association of Railway Superintendents of Bridges and Buildings varies from 9½ to 13 cents per square foot.

The Southern & Southwestern Railway Club—Eng. News, Jan. 16, 1896—recommends that round house floors be made of vitrified brick laid as follows: Make a bed surface of slag or chert about 18 inches thick, then put a coat of sand over slag, lay brick on edge and level them up by tamping. After this is done a coat of hot tar is applied which enters the space between the bricks and cements them together.

Wooden Floors.—Coal-tar or asphalt concrete makes the best foundation for a shop floor. If Portland cement is used, the planking will decay very rapidly unless the top of the concrete is mopped with coal-tar or asphalt. A floor laid by Pratt & Whitney Co., of Hartford, Conn., is described as follows: "In laying a basement floor about 18 years since of 10,000 square feet, 8,000 square feet were laid over coal-tar and pitch concrete in about equal proportions, and about 2,000 square feet were laid over cement concrete. The latter portion of the floor was removed in about ten years, the timbers and the plank being completely rotted out; while the other was in a perfect state of preservatoin and has continued so until the present time." The floor with tar concrete foundations was constructed as follows: "Excavation was made about one foot below the floor and six inches of coarse stone

was filled in, then five inches of concrete made of coarse gravel, coal-tar and pitch, and finally about one inch of fine gravel tar concrete. Before the concrete was laid, heavy stakes were driven about three feet apart to which the 4″ × 4″ floor timbers were nailed and leveled up. The concrete was then filled in around the floor timbers and thoroughly tamped. A layer of hot coal-tar was then spread on top of the concrete and the flooring was laid and nailed to the timbers. It is very essential that the gravel be perfectly dry before mixing and this is accomplished by mixing it with hot coal-tar. What is known as distilled or refined coal-tar must be used as that which comes from the gas house without being refined does not work in a very satisfactory manner."

The following paragraph is abstracted from Report No. V., Insurance Engineering Experiment Station, Boston, Mass:

"Floors over an air space or on cement are subject to a dry rot. Asphalt or coal-tar concrete is softened by oil, and the dust will wear machinery unless the concrete is covered by plank flooring. Floors made by laying sleepers on 6 inches of pebbles, tarred when hot, then 2 inches tarred sand packed flush with the top of the sleepers, and covered with a double flooring, have remained sound for 37 years. Double flooring at right angles can be laid on concrete without the use of sleepers. It is usually preferable to secure nailing strips to stakes 4 feet apart each way and driven to grade, concrete flush to top of strips, and lay 1½-inch flooring."

The floor shown in Fig. 135 was laid in an extensive shop on the Boston & Maine Railway. The earth was well compacted and brought to a proper surface and a 4-inch bed of coal-tar concrete put down in

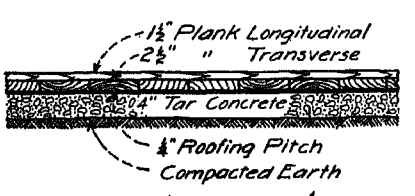


FIG. 135.

three courses. The stones in the lower course were to be not less than 1 inch in diameter. Stones in each course were well covered with tar before laying and were well tamped and rolled afterwards. The third and finishing course was composed of good clean sharp sand well dried, heated hot and mixed with pitch and tar in proper proportions. This was then carefully rolled and brought to a true level to fit a straight edge. On the finished surface of the foundation was spread a coating ¼-inch thick of best roofing pitch put on hot and into which the lower course of the plank was laid before the pitch cooled.

Care was taken to have the planks thoroughly bedded in the pitch and after laying, the joints were filled with pitch. If vacant spaces appeared under the plank, they were filled up with pitch by boring through the plank. The cost of this flooring was about 18 cents per square foot, using spruce lumber.

A cheap but serviceable floor may be made as shown in Fig. 136. The soil is excavated to a depth of 12 to 15 inches and cinders are filled in and carefully tamped. The flooring planks are nailed to the sills which are bedded in the cinders. The life of the plank flooring can be increased by putting a coating of slaked lime on top of the cinders.

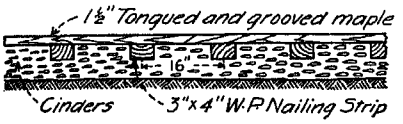


FIG. 136.

The floor shown in Fig. 137 was used in the factory of the Atlas Tack Company, Fairhaven, Mass., and needs no explanation.

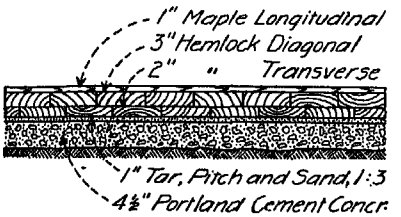


FIG. 137.

A very good floor for mills and factories is shown in Fig. 138.

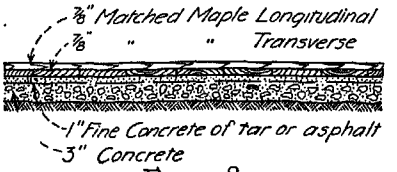


FIG. 138.

The pitch or asphalt will prevent the decay of the plank and will add materially to the life of the floor. Maple flooring makes the best wearing surface for floors and should be used if the cost is not prohibitive.

The floor shown in Fig. 139 was constructed as follows: Two-inch plank, matched and planed on one side, were laid on 3″ × 3″ chestnut joists. The surface of the cinders was kept 2″ away from the wood and

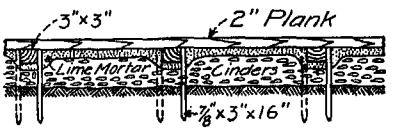


FIG. 139.

this space was filled with lime mortar. After the surface of the cinders had been graded, the 3″ × 3″ joists were held in place by stakes nailed to the joists about three feet apart. The lime mortar was then filled in around and slightly above the surface of the joists to allow for shrinkage. Before laying the floor a thin layer of slaked lime was spread over the surface. This floor in an eastern city cost about 85 cents per square yard; and has a life of 10 to 12 years.

Examples of Floors.—The floor of the Locomotive Shop of the A. T. & S. F. R. R., at Topeka, Kas., is as follows: The floor foundation is formed of 6 inches of concrete resting on the natural soil well tamped. On the concrete are laid 3″ × 4″ yellow pine stringers at 18-inch centers, the whole being covered with 2-inch No. 1 hard maple, surfaced on one side and two edges and milled for ¼″ × 1″ pine splines.

The flooring in the Great Northern Shops at St. Paul, Minn., is 3″ × 12″ plank on 6″ × 8″ sleepers bedded in 18 inches dry sand filling.

The floor of the locomotive shops of the Philadelphia and Reading R. R., is made of bituminous concrete on which are a solid course of 3″ × 8″ hemlock and a top wearing surface of 1⅛″ × 4″ maple.

The machine shop floors of the Lehigh Valley R. R. at Sayre Pa., are of concrete with a maple wearing surface in the high grade buildings and yellow pine in the others.

The Southern Railway has a vitrified brick floor in a round house at Knoxville, Tenn., which is giving good satisfaction. The cost of this floor was about $1.00 per square yard.

Shop floors for the American Locomotive Works, Schenectady, N. Y., are described in Engineering Record, May 30, 1903 ,as follows:

"On a sand fill was laid from 4 to 6 inches of 2½-inch broken stone, rammed dry and then flushed with about one gallon of hot tar for every square yard of floor. This course was covered with 2 inches of hot sand and tar mixed to the consistency of dry mortar, shoveled into place and thoroughly rammed to a level surface. Spiking strips made of 3″ × 4″ timbers were imbedded in the sand at about 3-ft. centers. To these strips were spiked 2-in. rough hemlock planks, which were in turn covered with transverse tongued and grooved ⅞-in. maple boards 4 inches wide."

Cedar Blocks form a neat, clean and durable floor. Care should be used where heavy jacking is to be done on wooden block floor that the blocks are not forced down through the plank foundation.

A cedar block floor in the Chicago Ave. round house of the C & N. W. R. R., laid on planks on a gravel foundation cost about 11 cents per square foot.

*A cedar block floor on the C. & E. I. R. R., laid directly on a gravel foundation cost 8 cents per square foot.

*A floor constructed of 6 to 8-inch blocks sawed from old bridge timbers, set on 2-inch hemlock plank, which in turn rested on 3 inches

*Reports Association of Railway Superintendents of Bridges and Buildings.

of dry gravel or sand, has been used on the Ashland division of the C. & N. W. R. R., and cost about 4 cents per square foot exclusive of the cost of the old timbers. This floor has proved to be quite satisfactory.

FLOORS ABOVE GROUND.—The type of floor used for the upper stories of mill buildings will depend upon the character of the structure and the use to which the floor is to be put. In fireproof buildings the floors should preferably be constructed of fireproof materials, although there is comparatively little risk from fire under ordinary conditions with a heavy plank floor. Where the load on the floor is very heavy some form of trough or buckled plate floor is very often used.

Timber Upper Floors.—Where steel floor beams are used the floor is often made by placing 2″ × 6″ or 2″ × 8″ planks on edge and spiking them together, the wearing surface being made of hard wood boards. Where there is much danger from fire this floor can be fireproofed by plastering it below with wire lath and hard plaster and by putting a layer of cement or lime mortar between the plank and the wearing surface. The upper surface is also sometimes finished with a wearing coat of cement or asphalt.

The standard floor recommended by the Boston Manufacturer's Mutual Fire Insurance Co., for mill buildings constructed of heavy timbers, calls for a layer of spruce plank, generally 3 inches thick, laid to cover two floor beam spaces and breaking joints every 3 feet; on this are laid 3 thicknesses of rosin sized paper, each layer being mopped with tar. The top floor is 1⅛-in. hard wood, preferably maple. The main beams are spaced 8 to 10 feet. "The floor is smoother if laid across the line of the plank in the under floor, but traveling loads are better distributed when moved in and out of the store house if the top floor is laid parallel to the lower plank."

Brick Arch Floor.—The brick arch floor shown in Fig. 140 was formerly much used in fireproof buildings and is still used to some extent in mills and factories. The arch is commonly made of a single

Brick Arch Construction
FIG. 140.

layer of brick about 4 inches thick, with a span of 4 to 8 feet and a center rise of preferably not less than 1/8 the span. The space above the brick arch is filled with concrete and a wearing floor is nailed to strips imbedded in the surface of the concrete. The most desirable span is from 4 to 6 feet. Tie rods are commonly placed at about 1/3 the height of the beam and are spaced from 4 to 6 feet apart. The thrust of the arch per lineal foot can be found by the formula

$$T = \frac{1.5\ W\ L^2}{R}$$

where T = thrust of arch in lbs. per lineal foot;
W = load on arch in lbs. per square foot;
L = span of the arch in feet;
R = rise of arch in inches.

The weight of this floor is about 75 lbs. per square foot.

Corrugated Iron Arch Floor.—The corrugated iron arch shown in Fig. 141 makes a very strong floor for shops and mills. The corrugated iron acts as a center for the concrete filling above it, and in connection with the concrete makes a composite arch. The corrugated iron or steel is ordinarily the standard 2½-inch corrugations, and the gages are Nos. 16, 18 or 20, depending upon the load and the length of span. The rise of the arch should not be less than 1-12 the span and

Corrugated Iron Arch
FIG. 141.

ter of the arch. Beams are spaced from 4 to 7 feet apart for this floor, and tie rods are used as in the brick arch floor.

Expanded Metal Floors.—The floor shown in Fig. 142 is constructed as follows: A wood centering is suspended from the beams, with the upper surface of the centering about 1 inch below the top of the beams, a layer of expanded metal is stretched across the beam in sheets and the concrete is spread over and tamped so that the expanded metal becomes imbedded in the lower inch of the concrete. The

FIG. 142.

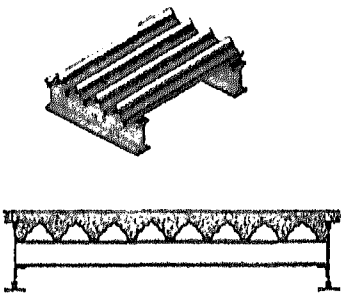
concrete is usually made of 1 part Portland cement, 2 parts sand and 6 parts cinders, and weighs 80 to 90 pounds per cubic foot. Beams were formerly spaced from 5 to 8 feet apart, but have recently been spaced much farther; spacings as wide as 18 to 20 feet having been successfully employed. Expanded metal with 3-inch mesh cut from No. 10 gage sheet steel is commonly used for floors, which are made from 3 to 5 or 6 inches thick. For the design of expanded metal and other forms of reinforced concrete floors, see the author's "The Design of Highway Bridges." The companies controlling the patents on this material will furnish gratis, data and tables for the design of expanded metal floors. Examples of floors may be found by consulting the Engineering News, Engineering Record, etc.

Expanded metal floors are also made as shown in Fig. 143. This type is adapted to very heavy floor loads. The arch should have a rise

FIG. 143.

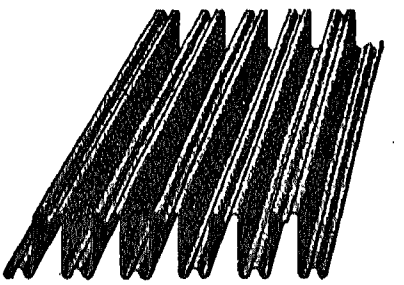
of 1-10 to ⅛ the span. Tests have shown that the steel work in expanded metal floors is not ordinarily affected by the cinder concrete. If care is used when erecting the floor to coat the expanded metal with a coating of Portland cement mortar before the metal has become rusted, the protection against corrosion will be almost perfect.

Roebling Floor.—The floor shown in (c) Fig. 144 consists of a wire cloth arch, stiffened by woven-in stiff steel rods ⅜ to 1-inch in diameter, at about 9-inch centers, which is sprung between the floor beams and abuts on the seat formed by the lower edge of the floor beam.



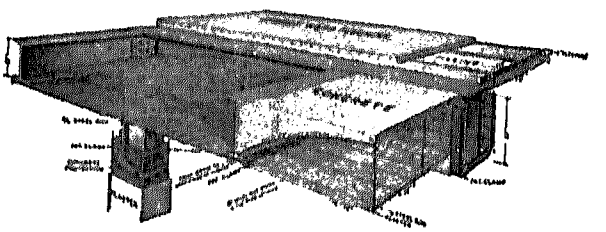
"BUCKEYE" FIREPROOF FLOORING.

(a)



MULTIPLEX STEEL PLATE FLOOR

(b)



Roebling Fire-proof System

(c)

(d)

FIG. 144.

On this wire centering Portland cement concrete is filled in and is finished with a wearing coat of cement or a wooden floor as shown. The beams are held in position by ¾ or ⅞-inch tie rods, placed from 4 to

6 feet apart. The concrete is commonly composed of 1 part Portland cement, 2½ parts sand and 6 parts clean cinders, and is laid with a thickness of not less than 3 inches over the crown of the arch. The weights and safe loads for floors constructed on this system are given in the manufacturer's catalog.

The Roebling Construction Company also makes a floor with flat construction as follows: A light framework is made of flat steel bars set on edge and spaced 16-inch centers, with ¼-turn at both ends where the bars rest on the steel beams; braces of half round iron are spaced at intervals to brace the bars. The Roebling standard lathing with ¼-inch steel stiffening ribs woven in every 7½ inches, is then applied to the under side of the bars and laced to them at each intersection. On this wire lathing cinder concrete from 3 to 4 inches thick is deposited.

"Buckeye" Fireproof Flooring.—The steel flooring shown in (a) Fig. 144 is manufactured by the Youngstown Iron & Steel Roofing Co., Youngstown, Ohio. This floor is made in two sizes, one for bridge floors and the other for building floors. The flooring shown in (a) is for buildings, is made in sections of four triangles each in lengths up to 10 feet, and will lay a width of 21 inches. Each triangle is 5¼ inches wide and 2½ inches deep. The flooring when complete with a concrete filling and a 1½-inch wearing surface will weigh from 32 to 35 pounds per square foot. The weights of the metal troughs laid in place are given in the following table:

WEIGHTS OF METAL TROUGHS 2½ INCHES DEEP BY 5¼ INCHES WIDE, INCLUDING SIDE LAPS.

GALVANIZED TROUGHS.		BLACK IRON TROUGHS.	
No. 16—386 lbs. per 100 square feet.		No. 16—363 lbs. per 100 square feet.	
No. 17—350 " " " " "		No. 17—327 " " " " "	
No. 18—313 " " " " "		No. 18—290 " " " " "	
No. 19—278 " " " " "		No. 19—254 " " " " "	
No. 20—241 " " " " "		No. 20—218 " " " " "	
No. 22—204 " " " " "		No. 22—181 " " " " "	
No. 24—168 " " " " "		No. 24—145 " " " " "	

The safe loads in addition to the weight of the floor as given in the manufacturer's catalog are given in the following table:

SAFE LOADS FOR "BUCKEYE" FIREPROOF FLOOR.

Span.	No. 18 Gage.	No. 20 Gage.	No. 22 Gage.	No. 24 Gage.
3 ft. 0 in.	1050 lbs.	800 lbs.	580 lbs.	450 lbs.
3 " 6 "	820 "	570 "	425 "	320 "
4 " 0 "	675 "	425 "	315 "	230 "
4 " 6 "	570 "	320 "	240 "	190 "
5 " 0 "	475 "	250 "	180 "	135 "
5 " 6 "	200 "	200 "	140 "	

Multiplex Steel Floor.—The steel flooring shown in (b) Fig. 144, is manufactured by the Berger Mfg. Co., Canton, Ohio. This floor is made with corrugations from 2 to 4 inches deep and of Nos. 16, 18, 20 and 24 gage steel. The triangles are filled with concrete and the floor is given a cement finish, or is covered with a wooden wearing surface. Tables of safe loads for Multiplex Steel Floor are given in the manufacturer's catalog, and are larger than for the "Buckeye" Flooring for the same span.

Ferroinclave.—Ferroinclave is fully described in Chapter XIX. It has been used by the Brown Hoisting Company for floors as well as for roofing and siding. It should make a very satisfactory flooring where light loads are to be carried.

Corrugated Flooring.—Corrugated flooring or trough plates shown in (a) and (b) Fig. 145, are used for fireproof floors where extra heavy loads are to be carried in mill buildings, in train sheds and for bridge floors. The troughs are filled with concrete, which is given a finishing coat of cement and sand or is covered with a plank floor; the planks being laid directly on the plates or spiked to spiking pieces imbedded in the filling. The details, weights and safe loads for corrugated plates are given in Pencoyd Iron Works' handbook, in Carnegie Steel Company's handbook, and in Trautwine's Pocket-book. Details of corrugated plates are also given in the American Bridge Company's "Standards for Structural Details."

Corrugated flooring or trough plates are usually very hard to get and the Z-bar and plate floor shown in (c) Fig. 145, and the angle and

FIG. 145.

plate floor shown in (d) are substituted. The details, weights and safe loads for Z-bar and plate flooring are given in the handbooks above named. Angle and plate flooring is made of equal legged angles and plates, and the safe loads are not given in the handbooks but must be calculated. The moment of inertia, I, of a section of flooring containing two angles and two plates is given by the formula

$$I = 2I' + 2Ad^2 + 2I''$$

where I' = moment of inertia of one angle about an axis through the center of gravity of the angle parallel to the neutral axis of the flooring;

A = area of one angle;

d = distance from center of gravity of the angle to the neutral axis of the flooring;

I'' = moment of inertia of the plate about the neutral axis.

The properties of the angles required in the calculations may be obtained from the handbooks, and I'' is equal to one-half the sum of the moments of inertia of the plate about its long and its short diameter—since the sum of the moments of inertia about any pair of rectangular axes is a constant.

Buckled Plates.—Buckled plates are made from soft steel plates from 3 to 5 feet wide, and are from ¼ to 7-16 inches thick. Buckled plates are made in lengths having from one to 15 buckles or domes in one plate. Buckles vary in depth from 2 to 3½ inches, however, different depths should not be used in the same plate. Buckled plates are usually supported along the edges and the ends and are bolted to the floor beams. The details, weights and safe loads are given in the handbooks named above and in the Passaic Steel Company's handbook. The

buckled plates are covered with concrete, which is given a finishing coat of cement or is covered with plank flooring.

Steel Plate Flooring.—Fireproof floors around smelters, etc., are often made of steel plates. Flat steel plates do not make a very satisfactory floor for the reasons that the plates will bulge up in the center when fastened around the edges, and because they become dangerously smooth.

Neverslip Wrought Steel Floor Plates are made from 24" × 72" to 36" × 120", and from 3-16" to 1" thick. These plates are designed to take the place of the cast iron checkered plates formerly used for floors, weigh about 50 per cent less and last much longer. The stock sizes, weights and safe loads for Neverslip floor plates are given in the Stock List of the Scully Steel Co., Chicago. These lists are issued about six times a year and will be sent free upon request.

Thickness of Timber Flooring.—The thickness of white pine and spruce flooring for different spans and loads is given in Table XXXIIIa. The following fibre stresses were used in calculating the thickness: Transverse bending, 1000 lb.; end bearing, 1000 lb.; bearing across the fibre, 200 lb.; shear along the fibre, 100 lb.

TABLE XXXIIIa.

THICKNESS OF SPRUCE AND WHITE PINE PLANK FOR FLOORS.

Span in Feet.	Thickness, in Inches, for Various Loads per Square Foot of Plank.																
	lb. 30	lb. 40	lb. 50	lb. 75	lb. 100	lb. 125	lb. 150	lb. 175	lb. 200	lb. 225	lb. 250	lb. 275	lb. 300	lb. 325	lb. 350	lb. 375	lb. 400
4.	0.9	1.1	1.2	1.5	1.7	1.9	2.1	2.2	2.4	2.5	2.7	2.8	2.9	3.1	3.2	3.3	3.4
5..... ..	1.2	1.4	1.5	1.9	2.1	2.4	2.6	2.8	3.0	3.2	3.4	3.5	3.7	3.8	4.0	4.1	4.3
6.	1.4	1.6	1.8	2.2	2.6	2.9	3.1	3.4	3.6	3.8	4.0	4.2	4.4	4.6	4.8	4.9	5.1
7.....	1.7	1.9	2.1	2.6	3.0	3.3	3.7	3.9	4.2	4.5	4.7	4.9	5.2	5.4	5.6	5.8	6.0
8.....	1.9	2.2	2.4	3.0	3.4	3.8	4.4	4.5	4.8	5.1	5.4	5.7	5.9	6.1	.....	.....	.....
9.....	2.1	2.5	2.7	3.4	3.9	4.3	4.7	5.1	5.4	5.8	6.1	.....	.....	.....	.....	.....	.....
10.....	2.4	2.7	3.1	3.7	4.3	4.8	5.2	5.6	6.0	.....	.....	.....	.....	.....	.....	.....	.....
11.....	2.6	3.0	3.4	4.1	4.7	5.3	5.8	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
12.....	2.9	3.3	3.7	4.5	5.2	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
13.....	3.1	3.6	4.0	4.9	5.6	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
14.....	3.4	3.9	4.3	5.3	6.1	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....

For yellow pine use nine-tenths of the above thicknesses.

CHAPTER XXIII.

Windows and Skylights.

Glazing.—For glazing windows and skylights, two substances, glass and translucent fabric are in common use.

GLASS.—The principal kinds of glass used in windows and skylights are (1) plane or sheet glass; (2) rough plate or hammered glass; (3) ribbed or corrugated glass; (4) maze glass; (5) wire glass—glass with wire netting pressed into it; (6) ribbed wire glass; and (7) prisms.

(1) *Plane Glass.*—Plane or common window glass is technically known as sheet or cylinder glass. It is made by dipping a tube in molten glass and blowing the glass into a cylinder, which is then cut and pressed out flat. Without regard to quality sheet glass is divided according to thickness into "single strength" and "double strength" glass. Double strength glass is ⅛-inch thick while single strength glass is about 1-16-inch thick. In mill buildings, lights larger than 12″ × 14″ are usually made of double strength glass. With reference to quality sheet glass is divided into three grades AA, A, and B. The AA is the best quality, the A is good quality while the B is very poor. The B grade is suitable only for stables, cellars, etc. For residences, offices, and similar purposes nothing poorer than AA should be specified. The A grade does very well for ordinary mills, although the AA grade should be used if practicable.

(2) *Plate Glass.*—Plate glass is made by casting and not by blowing, and is finished by grinding and polishing on both sides until a smooth surface is obtained. It is usually ¼ or 3-16 inches thick. The price depends upon the size of the plate and the quality of the glass. The rough plate glass used in mills is not finished as carefully as for glass fronts, and it may contain many flaws that would not be allow-

able in the former case. The roughened surface of the glass prevents the entrance of direct sunlight and does away with the use of sun-shades. The only value of rough plate glass is in softening the light, the loss of light in passing through it being very great.

(3) *Ribbed or Corrugated Glass.*—Ribbed or corrugated glass is usually smooth on one side and has 5, 7, 11 or 21 ribs on the other side, (a) Fig. 146. It varies in thickness and shape of ribs. "Factory ribbed" glass with 21 ribs to the inch is distinctly the most effective of the ribbed glasses.

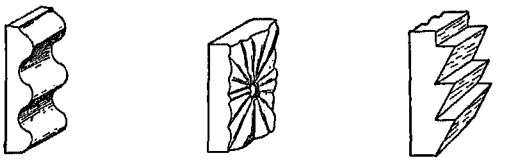


FIG. 146.

(4) *Maze Glass.*—Maze glass has one side smooth and has a raised pattern on the other side roughening practically the entire surface, (b) Fig. 146. It is quite effective.

(5) *Wire Glass.*—Wire glass is made by pressing wire netting into the molten glass. It is made either plane or with ribs or prisms on one side. Wire glass is injured but not destroyed by the action of fire and water, and is now accepted by insurance companies as fire-proof construction.

(7) *Prisms.*—Prisms are made in small sections which are set in a frame of lead or other metal, or are made in sheets as shown in (c) Fig. 146. Luxfer sheet prisms, manufactured by the American Luxfer Prism Co., Chicago, will be cut in any size desired up to 84 inches wide (parallel with the saw teeth) by 36 inches high.

Diffusion of Light.*—The light entering a room through a window or skylight comes for the most part from the sky and has, there-

*Report No. III. Insurance Engineering Experiment Station, Boston, Mass.

fore, a general downward direction, varying with the time of day and the position of the window. The portion of the room which receives the most light ordinarily is the floor near the windows, but if we interpose a dispersive glass in this beam the light will no longer fall to the floor but will be spread out into a broad divergent beam falling with nearly equal intensity on walls, ceiling and floor. There is of course no gain in the total amount of light admitted, the light being simply redistributed, taking up from the floor that which fell there and was comparatively useless, and sending it where it is of more service.

Experiments have shown that the diffusion of light in a room lighted by means of windows or skylights depends upon the kind and position of the glass used. The relative intensity of the light admitted in per cents of the light outside the window for plane glass, factory ribbed glass, Luxfer and canopy prisms is shown in Fig. 147*.

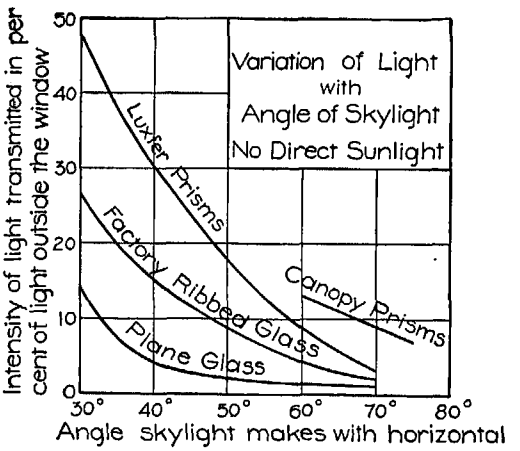


FIG. 147.

Fig. 147 shows a great increase in efficiency of factory ribbed glass and prisms as the sky angle diminishes.

*Report No. III. Insurance Engineering Experiment Station, Boston, Mass.

The equivalent areas required to give the same intensity of light with the kinds of glass shown in Fig. 147, are given in Table XXIV for sky angles of 30° and 60°.

TABLE XXIV.

EQUIVALENT AREAS FOR DIFFERENT KINDS OF GLASS.

Kinds of Glass.	Angle Skylight makes with the Horizontal.	
	30°	60°
Plane	100 sq. ft.	100 sq. ft.
Factory Ribbed	25 " "	40 " "
Luxfer Prisms	17 " "	30 " "
Luxfer Canopy Prisms	13 " "	

The American Luxfer Prism Co., recommends that Luxfer prisms be set at an angle of about 57 degrees with the vertical when used in skylights.

Relative Value of Different Kinds of Glass.—Ground glass is of little value except as a softening medium for bright sunlight. It becomes opaque with moisture and makes an undesirable window glass. Roughened plate glass has very little value as a diffusing medium. Of the ribbed glasses, the factory ribbed glass with 21 ribs to the inch gives the widest and most uniform distribution and is distinctly the best. There is no apparent gain in corrugating both sides. Ribbed wire glass is about 20 per cent less effective than the factory ribbed glass. When a glass of a slightly better appearance than the factory ribbed glass is wanted the maze glass is the best; the raised pattern imprinted on the back of this glass giving wide diffusion, especially in bright sunlight. The prisms are very much more effective than any of the glasses mentioned above, but their cost prevents their use under ordinary conditions.

Kind of Glass to Use.—Where the amount of skylight is large and the light is not obstructed by buildings plane glass is very satisfactory. Where a superior light is desired, or where the skylight area is less than ample, use factory ribbed glass in skylights and in the upper

panes of windows. Where the skylight area is very small, the light is obstructed, or a very superior light is desired, use prisms. Wire glass should be used where there is danger from fire and in skylights, where it removes the necessity of stretching wire netting under the glass to protect it and to prevent it from falling into the building when broken.

Placing the Glass.—Factory ribbed glass is somewhat more effective if the ribs are placed horizontal, but the lines of light deflected from the horizontal ribs may become injurious to the workmen's eyes and it is now the custom to set the ribs vertical. Ribbed glass should have the ribs on the inside for ease in keeping it clean, and where double glass is used the ribs should face each other and be crossed. Care should be used in setting thick wire glass in metal frames; the lower edge must bear directly on the frame, but the top and sides should fit loosely so that the differential expansion of the glass and frame will not crack the glass. Plane glass and small panes of other kinds of glass are set with glaziers' tacks and putty. In skylights and large windows some method must be used that will allow the glass to expand and contract freely and at the same time will be free from leakage. Several methods of glazing skylights without putty are shown in Fig. 148. Skylight bar (a) manufactured by Vaile & Young, Baltimore Md., is made of heavy galvanized iron and lead.

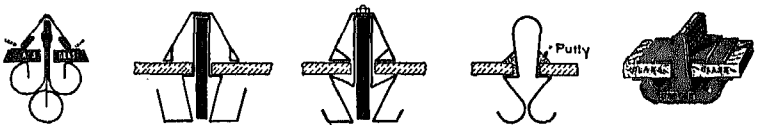


FIG. 148.

Bars (b) and (c) are made of zinc or galvanized iron, supported by a steel bar. Bar (c) is adapted to small panes of glass and is made of galvanized iron; it is made water tight by the use of putty. The skylight bars in Fig. 148, all have condensation gutters to catch the moisture that leaks through or forms on the inner surface of the glass.

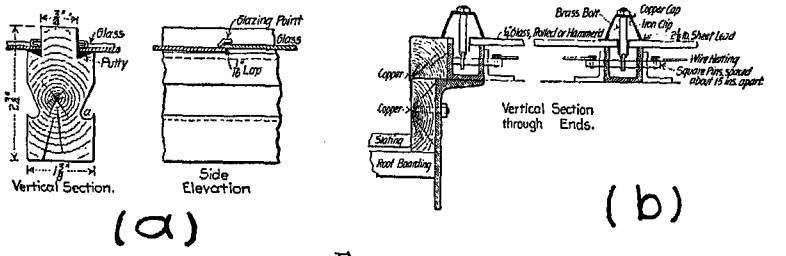


FIG. 149.

The glass in a large greenhouse at Edgely Pa., was secured to the sash-bars as shown in (a), Fig. 149. It will be seen that the glass is imbedded in putty on the under side only, and that any water that can possibly leak through between the bar and the glass will be caught in the drip trough "a", and be carried to the eaves. The lights are 16" × 24" and the sash-bars are spaced 24¾ ins., c. to c. The lights are held in place by two patent glazing points per light, driven in such a way as to prevent the glass from moving. The lights overlap but 1-16-in., the leakage having been found to be smaller and less liable to occur with this than with a larger lap.

The Paradigm system of glazing is shown in (b) Fig. 149. This system is in use in a large number of shops, among which the steam engineering buildings for the Brooklyn Navy Yard, described in Part IV, is one of the best examples. The patents for the Paradigm skylight are controlled by Arthur E. Rendle, New York.

Skylights are of two types; (1) box skylights covering a small area and placed on a curb raising the glass above the roof, and (2) continuous skylights usually placed in the plane of the roof. The glass used for skylights varies from ¼ to ⅛ inch thick and should preferably be wire glass. The glass used for skylights usually comes in sheets about 20 inches wide and up to 8 feet long.

The details of a box skylight manufactured by Vaile & Young, Baltimore, Md., is shown in Fig. 150.

Use of Window Shades.—Where factory ribbed glass is placed so as to throw light on the ceiling, screens or shades are seldom required,

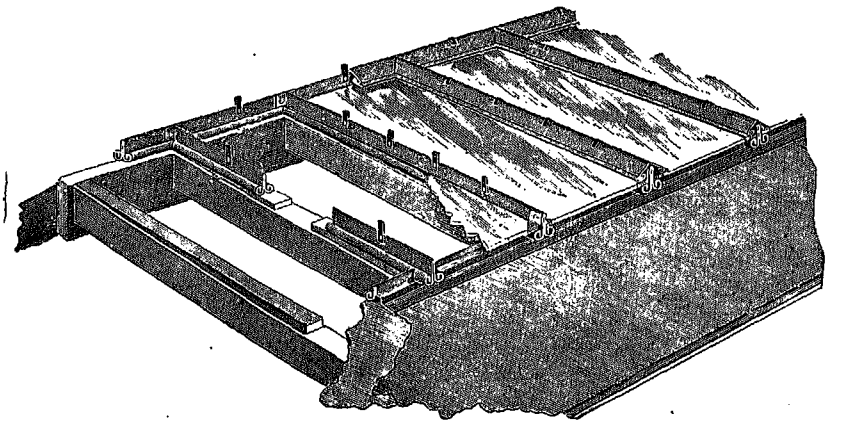


FIG. 150.

however, under ordinary conditions shades are necessary when bright sunlight strikes the window. The glass used in factory ribbed and rough plate glass as made in England is somewhat opaque, and the atmosphere is somewhat hazy, so that the use of shades in their shops is in most cases unnecessary. The glass made in this country is so clear and our atmosphere is so translucent that it has been found necessary to use shades where windows and shades are exposed to direct sunlight. The most effective and satisfactory shade is a thin white cloth, which cuts off about 60 per cent of the light.

Size and Cost of Glass.—The regular stock sizes of plane glass varies from 6 × 16 inches by single inches up to 24 × 30 inches, and above that by even inches up to 60 × 70 inches for double strength glass and 30 × 50 inches for single strength glass.

The weights of different thickness of glass, assuming 156 pounds as the weight of one cubic foot of glass are given in the following table:

WEIGHT OF GLASS PER SQUARE FOOT.

Thickness—in.	⅛	3-16	¼	⅜	½	⅝	¾	1
Weight—lbs.	1.62	2.43	3.25	4.88	6.50	8.13	9.75	13

The cost varies with the quality and the size, being about twice as much to glaze a given area with 30 × 36-inch lights as with 10 × 12-inch lights. The discounts given from the standard price list vary so

much that prices are of very little value except to give an idea of the relative cost of different sizes of glass and to serve as a basis for estimates.

In 1903 American window glass was quoted about as given in Table XXV.

TABLE XXV.

COST OF WINDOW GLASS IN CENTS PER SQUARE FOOT.

Size of Lights in Inches.	Single Strength.			Double Strength.		
	AA	A	B	AA	A	B
10 × 12	5.1	4.3	4.0	6.8	6.0	5.5
14 × 20	5.4	4.5	4.3	7.5	6.6	6.1
16 × 24	5.8	4.8	4.5	8.3	7.3	6.7
20 × 30	6.0	5.1	4.7	8.9	7.9	7.4
24 × 36	6.4	5.5	4.9	9.4	8.3	7.6

In 1903 the different kinds of glass were quoted in small quantities at the factory about as follows:

Wire glass ¼ inch thick.....23 cents per sq. ft.
Factory ribbed glass ⅛ inch thick..... 9 " " " "
Maze glass ⅛ inch thick.....12 " " " "
Maze glass 3-16 inch thick.....18 " " " "
Prismatic glass from 25 to 50 cents per sq. ft.

Refrax glass (sheet prisms) made by the Union Plate Glass Co., Limited, Pocket Nook, St. Helens, England, was quoted in 1903 as follows at the factory: Ordinary refrax glass ¼" thick with 5 prisms to the inch, cut to any size up to 60" × 90", 20 cents per sq. ft.; wired refrax glass 5-16 inch thick with 5 prisms to the inch, cut to any size up to 40" × 90", 25 cents per sq. ft.

Maltby prisms, made by Geo. K. Maltby, Boston Mass., 3-16" thick with 6 prisms to the inch costs about 25 to 30 cents per sq. ft.

Cost of Windows.—Windows with frames for mill buildings will cost from 15 to 25 cents per square foot, depending upon the size and quality of the sash, the size of the opening, and cost of glass and frames. In 1900 the cost was about 16 cents per square foot for D. S.

glass with box frames and sash, and 9 cents for S. S. glass with plank frames and sash. In 1900 skylights cost from 23 to 30 cents per square foot with D. S. glass. Windows are commonly estimated at 25 cents per square foot and skylights at from 40 to 50 cents per square foot in making preliminary estimates.

The American Luxfer Prism Company manufacture sheet prisms for factory purposes that can be cut to fit any opening up to 36″ × 84″. The cost of sheet prisms to fit ordinary windows is about 40 cents per square foot. The improved skylight prisms made by this company cost about $1.50 per square foot.

TRANSLUCENT FABRIC.—Translucent fabric consists of a wire cloth imbedded in a translucent, impervious, elastic material, probably made of linseed oil. The fabric may be bent double without cracking and is so elastic that changes due to temperature or vibrations do not affect it. If a sheet of translucent fabric is suspended and a fire applied to the edge, it will burn up leaving a carbonaceous covering on the wire. But if the edges are protected it will burn only with great difficulty. Live coals falling on skylights of this material will char and burn holes but will not set fire to the fabric. It is therefore practically fireproof.

Translucent fabric will not transmit as much light as glass, but makes a most excellent substitute therefore. It shuts off sufficient light so that the lighting is uniform throughout the shop and makes it possible for men to work directly under it without shading. Where one-quarter of the roof is covered with the fabric the lighting is practically perfect. The fabric should be washed with castile soap and warm water occasionally, and should be varnished every year or two with a special varnish furnished by the manufacturers. It is said to become less opaque with age. When properly cared for the fabric has been known to give good service for ten years. The fabric is manufactured in sheets 3′ 3″ wide and in lengths from 4′ 6″ to 9′ 0″. The framework for translucent fabric is the best made of wood. A standard frame for sheets 3′ 3″ × 6′ 3″ is shown in Fig. 151. The fabric must be stretched tight and carefully nailed around the edges of the sheet. The capped joint with metal cap shown in Fig. 151 is very

satisfactory as it holds the fabric tight, and will give slightly to accommodate changes in temperature.

Cost of Translucent Fabric.—The fabric costs from 13 to 15 cents per square foot at the factory at Quincy, Mass. The framework, freight and cost of laying will probably be as much more, making the entire cost of skylights from 25 to 30 cents per square foot.

Translucent fabric has been quite widely used and has given uniformly good results. It has been used recently in the A. T. & S. F. R. R. shops at Topeka, Kas.

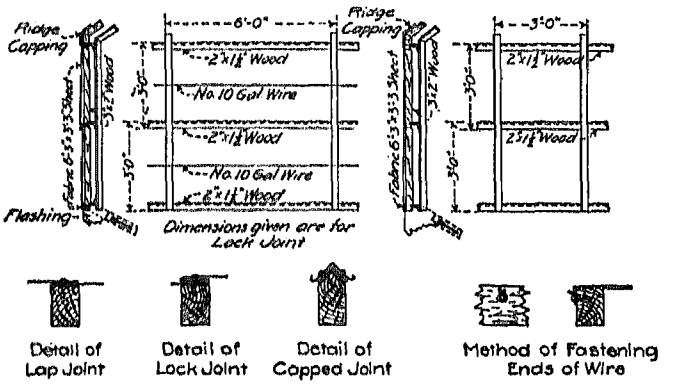


FIG. 151.

Double Glazing.—The condensation on the inner surface of glass can be prevented by double glazing the windows and skylights. Buildings with double glazing are also very much easier to heat than those with single glazing, the air space between the sheets of glass acting as an almost perfect non-conductor of heat.

Details of Windows and Skylights.—The details of windows in use in different sections of the country vary a great deal on account of the varied conditions. In buildings that have to be heated and ventilated through the windows at the same time, it is necessary to provide some means of opening and closing the windows quickly and easily; while in many other cases the sash can remain fixed. The author would call especial attention to the saving in fuel by the use of double glazing; the loss of heat through a double glazed skylight has been shown by

experiment to be only about one-half what it is through a single glazed skylight.

Details of windows for use in ordinary brick and stone walls can be found in books on architectural construction and will not be given here. A few of the best designs available for windows in buildings with corrugated steel, expanded metal and plaster, and similar walls, have been selected and are given on the following pages.

The different types of windows for buildings covered with corrugated steel siding as used by the American Bridge Company are shown in Figs. 152 to 156 inclusive.

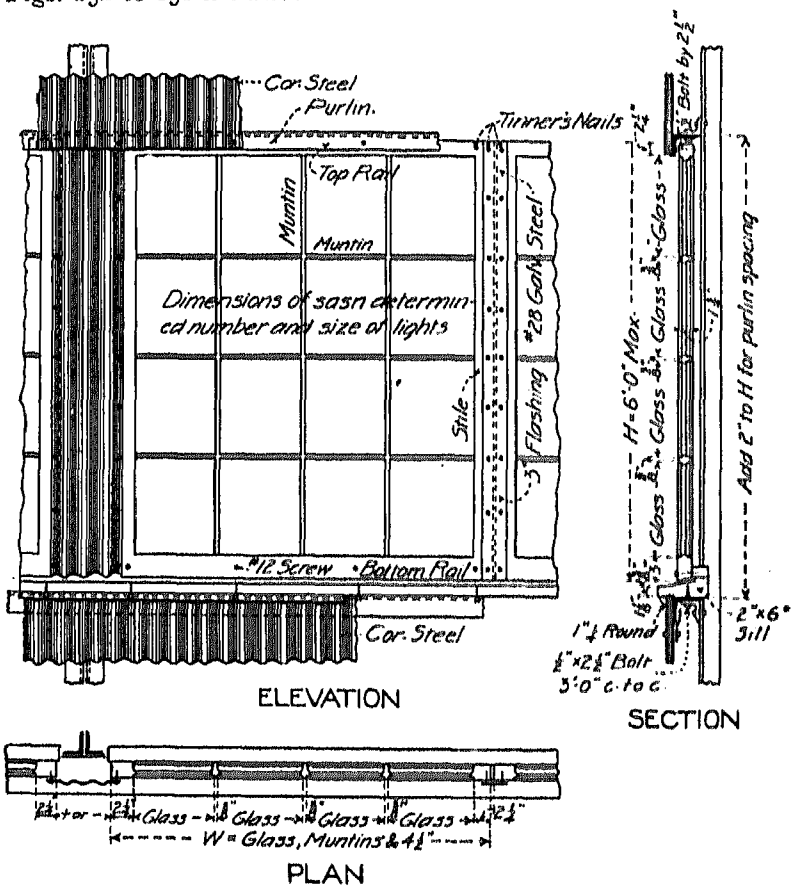


FIG. 152. DESIGN FOR A CONTINUOUS FIXED SASH WINDOW.

The sash frames are constructed of white pine and are glazed usually with A quality American glass. The common sizes of glass used in these windows are 10" × 12", 12" × 12", 10" × 14" and 12" × 14" single strength. For lights larger than 12" × 14", double strength glass is used. The window shown in Fig. 152 is used where light is desired without ventilation. This detail is used principally for monitor ventilators or for windows placed out of reach. Where it is desirable to obtain ventilation as well as light the window frame with sliding sash shown in Fig. 153 is used.

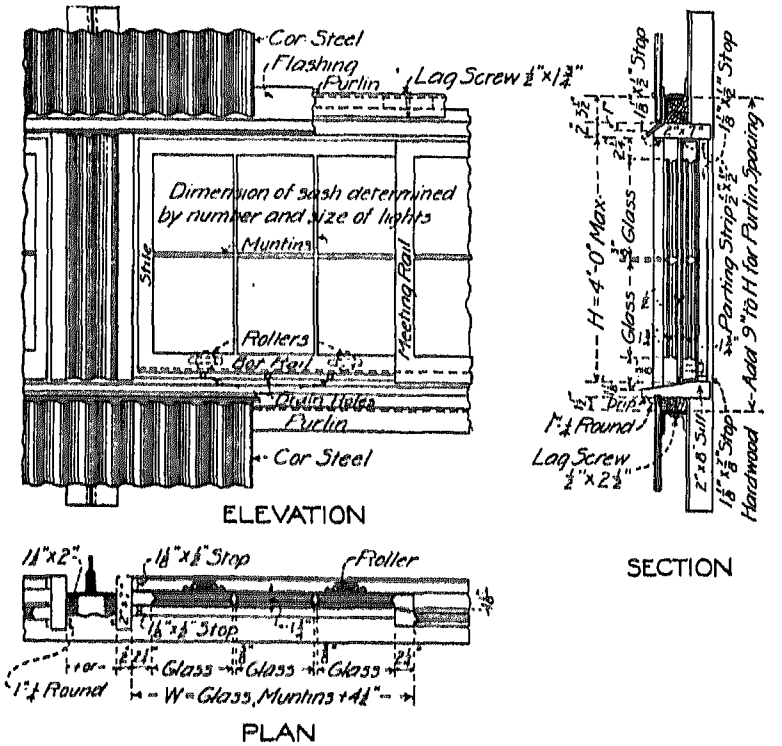


FIG. 153. DESIGN FOR WINDOW FRAME WITH SLIDING SASH.

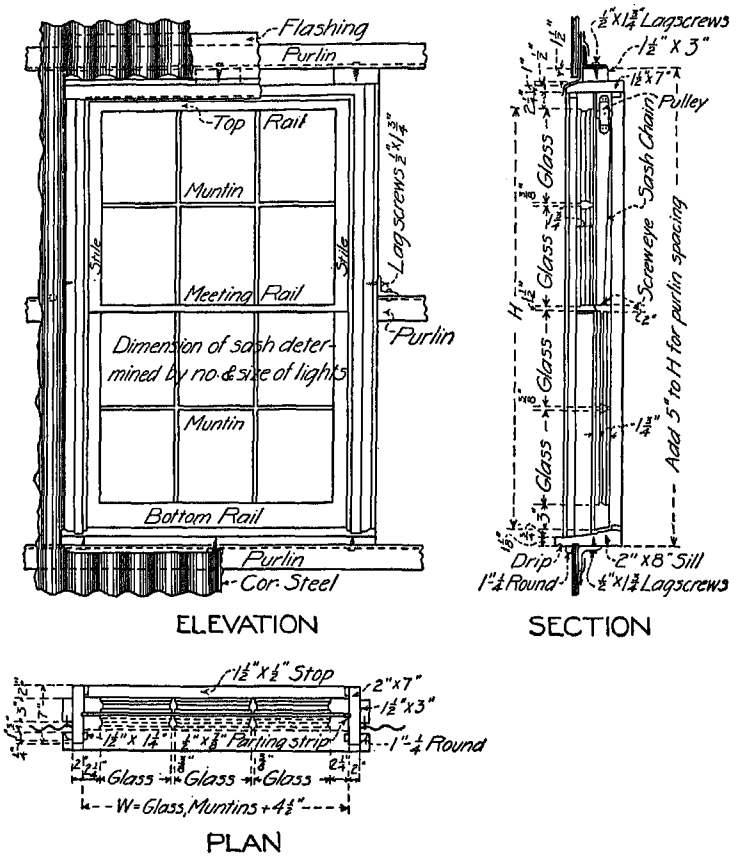


FIG. 154. DESIGN FOR WINDOW FRAME WITH COUNTERBALANCED SASH.

Amount of Light Required.—The amount of glazed surface required in mill buildings depends upon the use to which the building is to be put, the material used in glazing, the location and angle of the windows and skylights, and the clearness of the atmosphere. In glazing windows for mills and factories in which the determination of color is a necessary part of the work, care should be used to obtain a clear white glass for the reason that the ordinary commercial glass breaks up the light passing through it so that the determination of color is difficult.

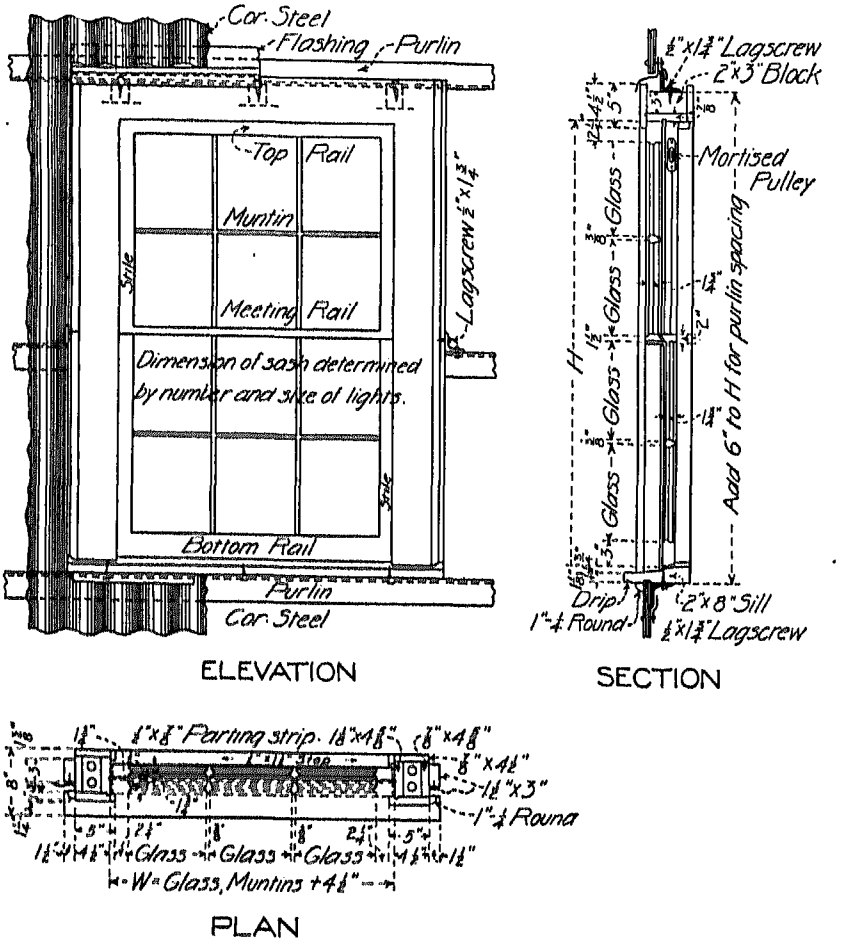


FIG. 155. DESIGN FOR WINDOW FRAME WITH WEIGHTED SASH.

It is common to specify that not less than 10 per cent of the exterior surface of ordinary mill buildings and 25 per cent of the exterior surface of machine shops and similar structures shall be glazed. One-half of the glazing is usually required to be in the roof in the form of skylights. With translucent fabric it has been found that the lighting is good where 25 per cent of the roof is glazed.

The present tendency in shop and factory design is to make as

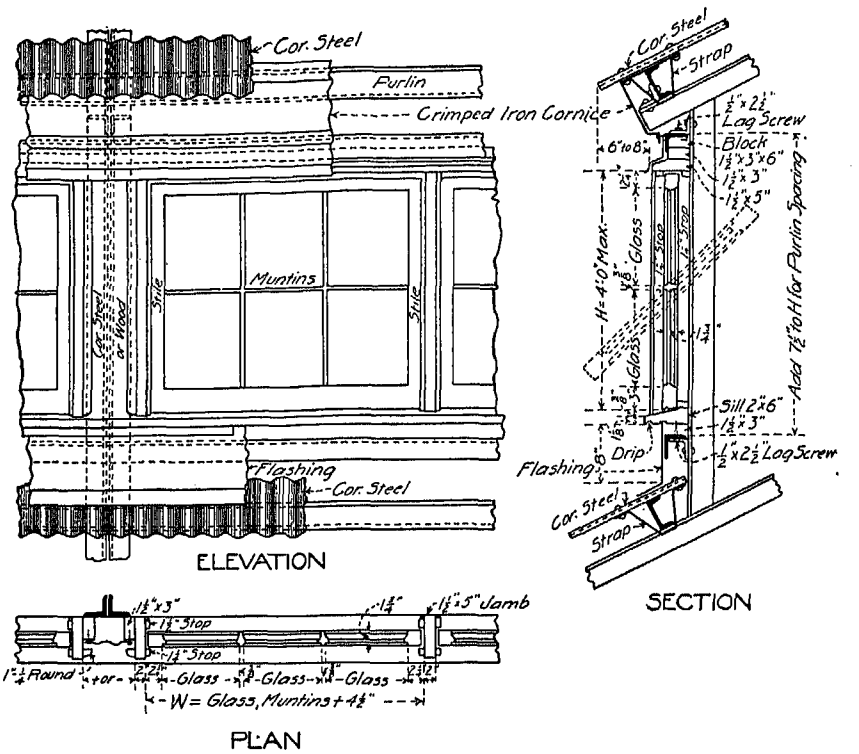


FIG. 156. DESIGN FOR WINDOW FRAME WITH SWINGING SASH.

much of the side walls and roof of glass as possible; the danger of leakage around and through skylights has prevented many from making use of skylights, although with the present methods of glazing there is no reason why any leakage should occur. The shops of the Grant Tool Company, at Franklin, Pa., shown in Fig. 157, is a good illustration of side wall lighting, while the steam engineering buildings for the Brooklyn Navy Yard, described in Part IV, is a good illustration of side wall and skylight lighting. The A. T. & S. F. shops described in Part IV, is a good illustration of side wall, skylight and saw tooth roof lighting combined.

The Central Railway of New Jersey shops at Elizabeth, N. J., have skylights made of translucent fabric in the different buildings in

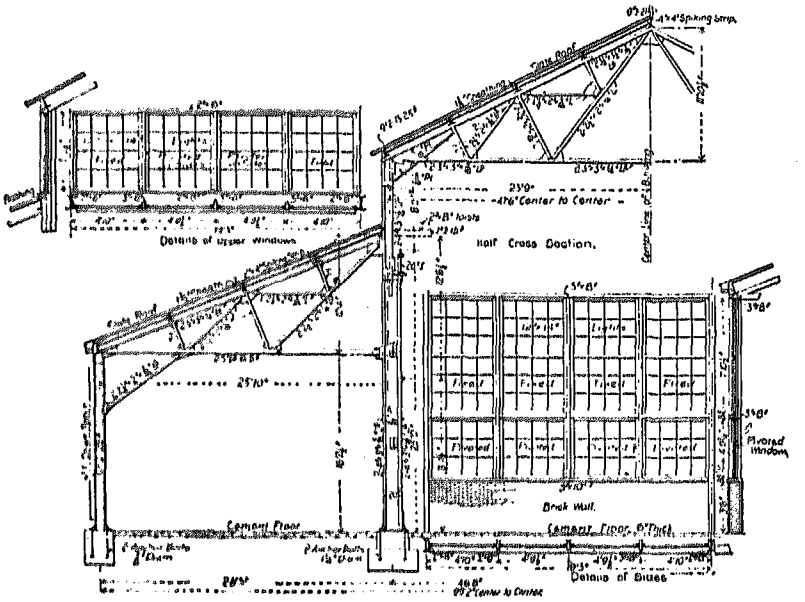


FIG. 157. SHOPS OF THE GRANT TOOL WORKS, FRANKLIN, PA.

per cents of the entire roof surface as follows: Blacksmith shop 30 per cent; machine shop 36 per cent; and paint and repair shop 55 per cent.

The Lackawanna and Western Railway blacksmith shop has 13 square feet of skylight per 100 square feet of floor area.

In the Great Northern Railway shops at St. Paul, Minn.,—Railway Gazette, June 16, 1903—all skylights have ¼-inch ribbed glass, below which is double strength window glass. Suitable drainage is provided for the moisture which collects on the upper surface of the latter. Wire netting is stretched under the skylights to prevent broken glass from falling into the shops. The walls are supplied with windows set at 12 feet centers, 55 panes to each sash.

The skylights of the A. T. & S. F. R. R. shops at Topeka, Kas., are made of translucent fabric, about 20 per cent of the roof surface being fabric.

The skylights of the machine shop of the Chicago City Railway are made of wire glass, about 35 per cent of the roof being glass.

The machine shop of the Lehigh Valley Ry., at Sayre, Pa., will have the side windows of plane glass. The locomotive shop will have factory ribbed glass in the side windows and wire glass in the roof and monitor skylights.

About 25 per cent of the roof of the St. Louis Train Shed is skylight.

In the American Car and Foundry Company's shop at Detroit, about 27 per cent of the exterior surface is ribbed glass.

Fully 60 per cent of the exterior surface of the Steam Engineering Buildings for the Brooklyn Navy Yard is of glass.

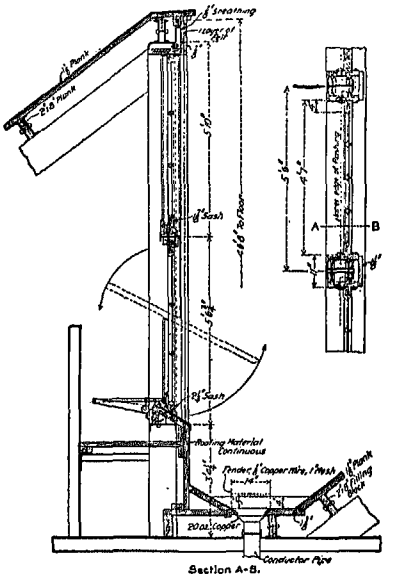


FIG. 157a. DETAILS OF SAW TOOTH WINDOWS FOR P. & L. E. R. R. SHOPS. SEE FIG. 84d.

Experience with Skylight Construction for Railway Train Sheds.—In 1904 a committee was appointed by a prominent eastern trunk line to investigate and report on train shed skylight failures and their remedy. The committee examined the Broad Street Station train shed, Camden and Reading Terminal train sheds, and other structures in Philadelphia; the train shed at Jersey City; the North German Lloyd Steamship piers at Hoboken; the N. Y. C. & H. R. R. R. train shed, the Macey Building, and other structures in New York; the South Union, North Union, and Back Bay stations, and the Charlestown Navy Yard in Boston; the Union Station, and the Pittsburg & Lake Erie station in Pittsburg; and the Westinghouse Shops in East Pittsburg. A complete report is printed in Engineering News, April 21, 1904.

The conclusions of the committee were as follows:

1. That gas and smoke from locomotives, because of its influence on the metal framework of skylights, is the primary cause of the breakage of glass.

2. That the contraction and expansion of the metal frame is also a serious cause of breakage where the glass is tightly fitted in the frames.

3. From the testimony elicited and from personal observations, we find the percentage of breakage in ribbed, hammered, and wire glass is about equal. We do not find that the breakage of wire glass results from any internal stress being set up by the contraction and expansion of the wire within.

4. The larger sizes of glass break more readily than the smaller.

5. Glass set horizontally, or at an angle, breaks more readily than glass set vertically.

6. Wire netting hung under glass, from the effect of gases upon it, is unreliable.

7. Wire glass is most desirable, because when fractured, the wire will generally hold it in position until repairs can be made.

8. Steel bars, such as are used in skylights at Broad Street and Jersey City train sheds, because of the effect of the gases on same, are unsatisfactory.

9. Wooden bars, such as are used in skylights at the Jersey City train shed, are desirable, being unaffected by gases. We recommend a zinc cap in the place of the wooden cap.

10. We recommend that a zinc expansion bar be used with brass bolts in preference to wooden bars. We are led to this conclusion from the condition of the zinc cap on the skylights on the Jersey City train shed, which show no deterioration from effect of gas.

11. We recommend that the sizes of glass used in skylights should not exceed 24 × 36 inches.

12. We recommend for future construction and present train sheds, where same can be adopted, a monitor form of skylight, preferably placed parallel with the tracks, of large dimensions; set far enough apart so that one monitor will not obstruct the light of another; small sizes of glass, set in wooden or approved metal frames, frames set loose enough to overcome the contraction and expansion of the metal work of the shed.

13. We recommend ventilating the monitors at the top, the opening being covered with an umbrella shelter, also by putting on each side of the monitor an opening under the eaves above the glass the entire length.

CHAPTER XXIV.

VENTILATORS.

Ventilation.—Mill buildings are ventilated either by forced draft or by natural ventilation. Natural ventilation is usually sufficient, although forced ventilation is necessary in many factories and mills. The problem of ventilation is too large to consider fully in this place and the natural method of ventilation only will be discussed. The amount of air required depends on the use to which the building is to be put; a common specification for the ventilation of mill buildings being that ventilators shall be provided and located so as to ventilate the building properly, and shall have a net opening for each 100 square feet of floor space of not less than one-fourth square foot for clean machine shops and similar buildings; of not less than one square foot for dirty machine shops; of not less than four square feet for mills; and not less than six square feet for forge shops, foundries and smelters. Ventilators in high buildings are more effective than in low ones. The following table will give an idea of the effect of height on ventilation.*

Height above ground.	20′	30′	40′	50′	
Machine shop, sq. ft. per 100	⅞	¾	⅝	½	round vents.
Mills, " " " "	7	6	5	4	Louvre vents.
Forge shops, " " " "	9	8	7	6	Louvres or open vents.

Monitor Ventilators.—The openings in the clerestory of monitors are fitted with louvres, shutters or sash, or may be left entirely open. Louvres are made in many different ways, the Shiffler Louvres shown in Fig. 158, and the Berlin Louvres shown in Fig. 159, are in common use. The details of these louvres as made by various firms differ somewhat.

*Mill Building Construction, H. G. Tyrrell.

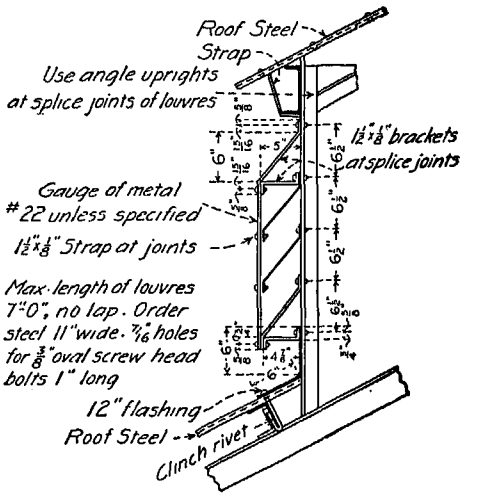


FIG. 158. SHIFFLER LOUVRES.

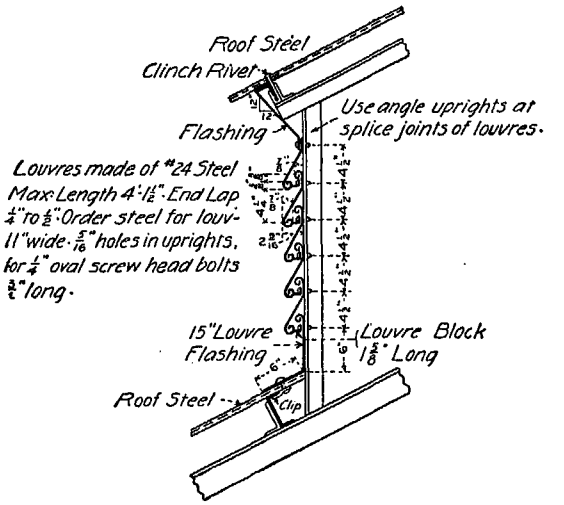


FIG. 159. BERLIN LOUVRES.

The details of the Shiffler louvres shown in Fig. 158, and of the Berlin louvres shown in Fig. 159 are those adopted by the American Bridge Company. The details of the louvres are shown in the cuts and need no further explanation.

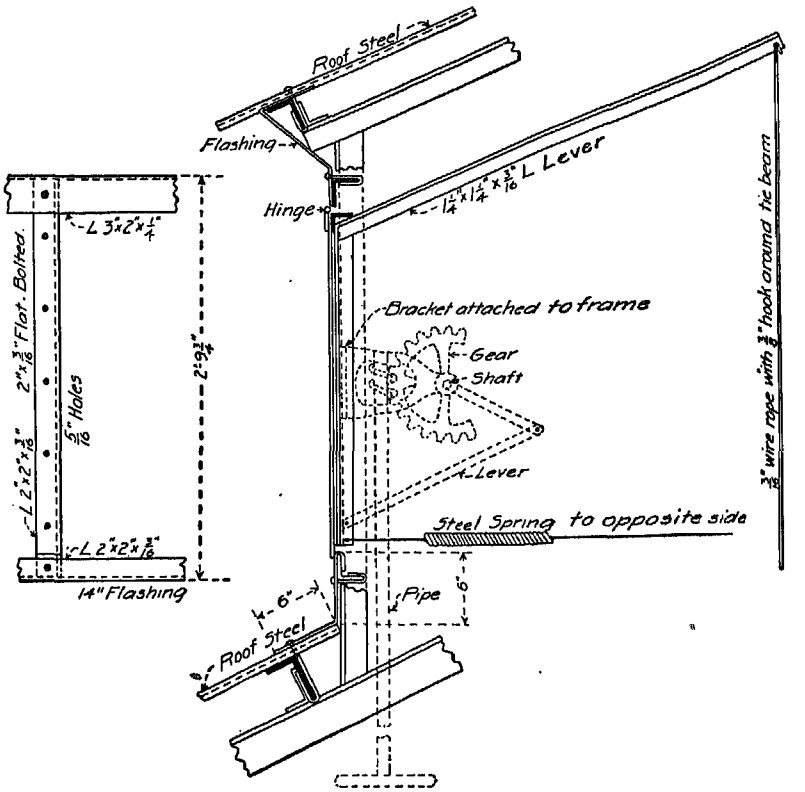


FIG. 160. HINGED MONITOR SHUTTER.

Details of a hinged shutter are shown in Fig. 160. The angle iron frame is covered with a corrugated iron covering. The shutters are made from 6 to 10 feet long, with two hinges for shutters 8 feet long and three hinges for shutters more than 8 feet long. Where shutters are to be glazed they are hung as in Fig. 156. The lever gear shown by the dotted lines is used in the better class of structures. This device can be used where the shutters are glazed if care is used in operating.

In smelters the clerestory of the monitor is often left entirely open or is slightly protected by self acting shutters. In the latter case the shutters are hinged at the bottom and are connected at the top with each other and with a counter-weight so that the shutter will ordinarily make

an angle of about 30 degrees with the vertical. A wind or a storm will close the windward shutter and open the leeward shutter wider. The eaves of the monitor are made to project, so that very little of the storm enters.

Cost.—The shop cost for louvres is ordinarily about 1 cent per pound. To this must be added the cost of the sheet steel and the cost of the framework and details. In 1900 louvres without frames cost about 25 cents per square foot.

Circular Ventilators.—Circular ventilators are often used for ventilating mill buildings in place of the monitors, and on buildings requiring a small area for ventilation. They are made of galvanized iron, copper or other sheet metal, and are usually placed along the ridge line of the roof.

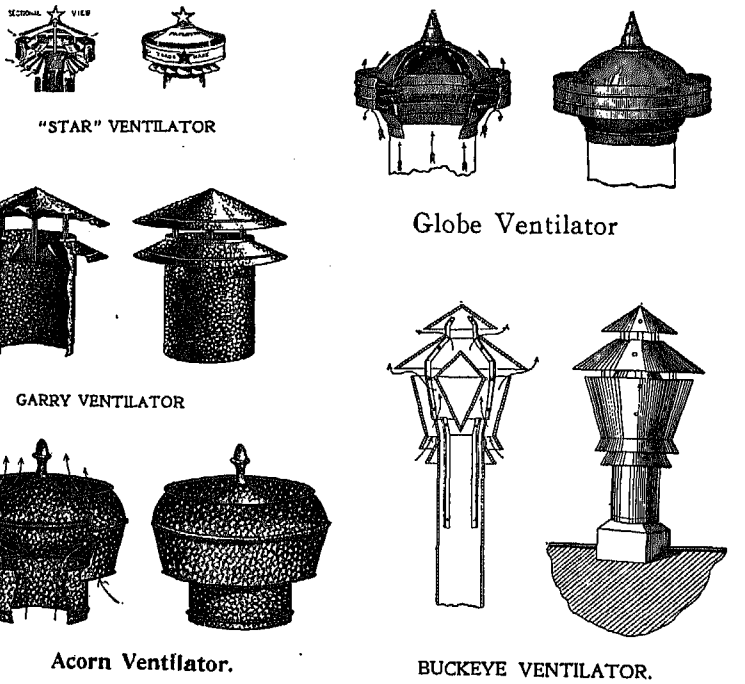


FIG. 161. CIRCULAR VENTILATORS.

There are many styles of circular ventilators on the market, a few of which are shown in Fig. 161. The Star ventilator made by Merchant & Co., Chicago, is quite often used and is quite efficient. It is made in sizes varying from 2 to 60 inches. In 1903 Star ventilators made of galvanized iron were quoted about as follows: 12-in., $2.00; 18-in., $6.75; 24-in., $10.00; 40-in., $45.00.

The Globe ventilator made by the Cincinnati Corrugating Company, Cincinnati, Ohio; the Garry ventilator made by the Garry Iron & Steel Roofing Co., Cleveland, Ohio; and the Acorn and Buckeye ventilators made by the Youngstown Iron & Steel Roofing Co., Youngstown, Ohio, are quite efficient and all cost about the same as the Star except the Garry ventilator, which is cheaper.

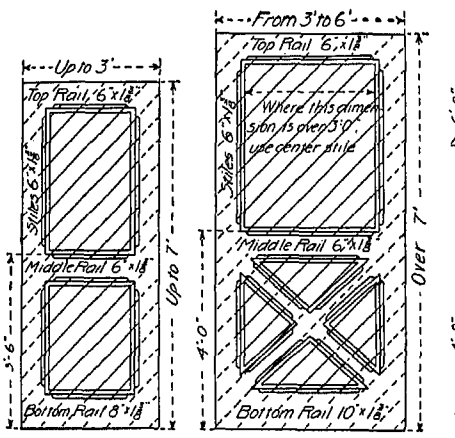
Home-made circular ventilators can be made that are quite as satisfactory as the patented ventilators and are much less expensive. In 1900, ten 36-inch circular ventilators cost $12.25 each, and two 24-inch circular ventilators cost $9.25 each in Minneapolis, Minn. The cost of the 24-inch ventilators was large on account of the small number made.

CHAPTER XXV.

Doors.

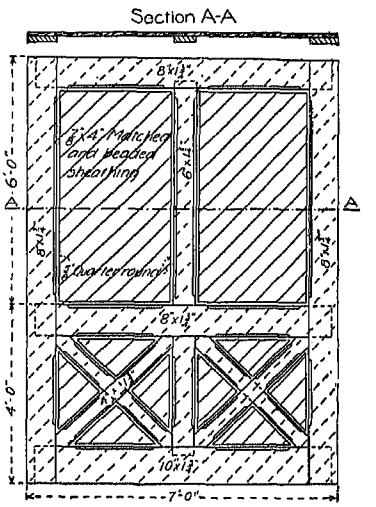
Paneled Doors.—For openings from 2′ 0″ × 6′ 0″ to 3′ 0″ × 9′ 0″ ordinary stock paneled doors are commonly used. The stock doors vary in width from 2′ 0″ to 3′ 0″ by even inches and in length by 4″ to 6″ up to 7′ 0″ for 2′ 0″ doors, and 9′ 0″ for 3′ 0″ doors. Stock doors are made 1⅜ and 1¾ inches thick, and are made in three grades, A, B and C; the A grade being first class, B grade fair and C grade very poor. Paneled doors up to 7 feet wide and 2¼ inches thick can be obtained from most mills by a special order.

Wooden Doors.—Wooden doors are usually constructed of matched pine sheathing nailed to a wooden frame as shown in Fig. 162 and Fig. 163.



Swing Wooden Doors

FIG. 162.



Sliding Wooden Door

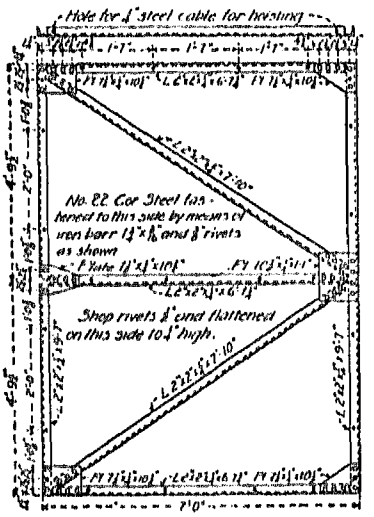
FIG. 163.

a wooden sliding door in Fig. 163. These doors are made of white pine. Doors up to four feet in width should be swung on hinges; wider doors should be made to slide on an overhead track or should be counterbalanced and raise vertically. Sliding doors should be at least 4 inches wider and 2 inches higher than the clear opening.

"Sandwich" doors are made by covering a wooden frame with flat or corrugated steel. The wooden framework of these doors is commonly made of two or more thicknesses of 7/8-inch dressed and matched white pine sheathing not over 4 inches wide, laid diagonally and nailed with clinch nails. Care must be used in handling sandwich doors made as above or they will warp out of shape. Corrugated steel with 1¼-inch corrugations makes the neatest covering for sandwich doors.

For swing doors use hinges about as follows: For doors 3′ × 6′ or less use 10-inch strap or 10-inch T hinges; for doors 3′ × 6′ to 3′ × 8′ use 16-inch strap or 16-inch T hinges; for doors 3′ × 8′ to 4′ × 10′ use 24-inch strap hinges.

Steel Doors.—Details of a steel lift door are shown in Fig. 164. This door is counterbalanced by weights and lifts upward between ver-

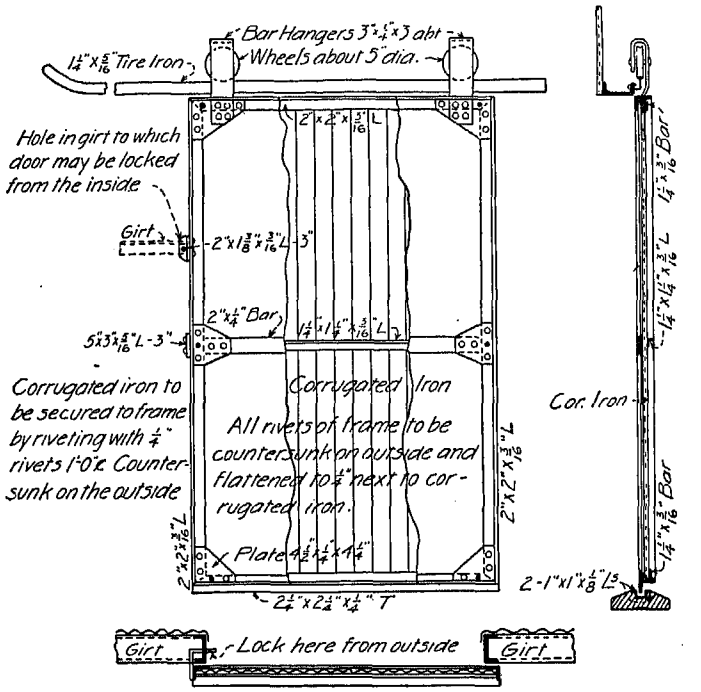


Steel Lift Door

FIG. 164.

tical guides. This door was covered with corrugated steel with 1¼-inch corrugations as described in the cut.

Details of a steel sliding door are shown in Fig. 165. This door is made to slide inside the building and swing clear of the columns. Where the columns are so close together that there is not room enough for the door to slide the entire length of the opening, it should be placed on the outside of the building. The track and hangers shown



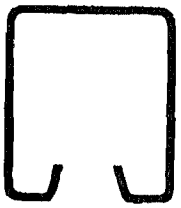
Steel Sliding Door

FIG. 165.

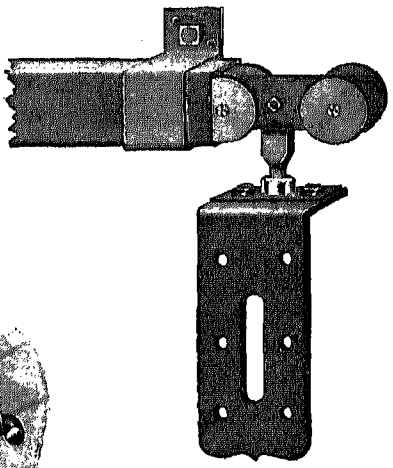
make a very satisfactory arrangement; however there is a tendency for the wheels to jump the track unless the grooves in the wheels are made very deep.

There are quite a number of patented devices on the market for hanging sliding doors. The Wilcox trolley door hanger, track and

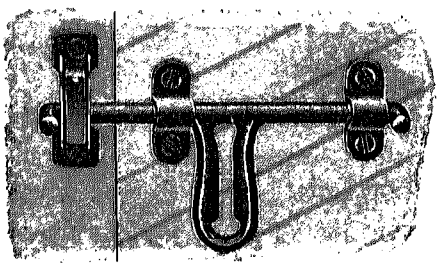
bolt latch shown in Fig. 166, are efficient and are quite generally used. The prices of the door fixtures shown in Fig. 166 are about as follows: door hangers, $2.25 to $3.00 per pair; steel track, 10 to 25 cts. per ft.; clips, 15 to 25 cts. each; door latch, $1.00, f. o. b. the factory at Aurora, Ill. Discounts for this and several other well known makes of door fixtures are given each week in the Iron Age, New York, and the list prices are given in the manufacturer's catalogs.



Wilcox O. K. Steel Track



Wilcox Trolley Door Hanger



Wilcox Gravity Door Bolt and Latch

FIG. 166.

Cost of Doors.—Stock panel doors cost $1.50 to $5.00 each, depending upon the grade, size and conditions. The details of steel doors vary so much that it is necessary to make detailed estimates in each case. The shop cost of the framework is often quite high and may run as high as 3 or 4 cts. per pound. The wooden frames for sandwich doors cost from 20 to 25 cts. per square foot. The cost of hinges, bolts, etc., required for doors can be found by applying the discounts given in the Iron Age to the list prices given in the standard lists (see Chapter XXVIII).

CHAPTER XXVI.

Shop Drawings and Rules.

SHOP DRAWINGS.—The rules for making shop drawings in use by the American Bridge Company are given in their Standards for Structural Details, and are reprinted in part, in Roofs and Bridges, Part III, by Merriman and Jacoby. The following rules are essentially those in common use by bridge companies, for mill buildings and warehouses.

Make sheets for shop details 24 by 36 inches, with two border lines, ½ and 1 inch from the edge, respectively. For mill details use special beam sheets. The title should come in the lower right hand corner, and should contain the name of the job, the contract number, and the initials of the draftsman and checker.

Detail drawings should be made to a scale of ¾ to 1 inch to the foot. Members should be detailed as nearly as practicable in the positions in which they occur in the structure. Show all elevations, sections, and views in their proper positions. Holes for field connections should always be blackened. Members that have been cut away to show a section, may be either blackened or cross-hatched. Members, the ends of which are shown in elevation or plan, should be neither blackened nor cross-hatched. Holes for field connections should be located independently, and should be tied to a gage line of the member. When metal is to be planed, the ordered and finished thickness should be given.

In making shop drawings for mill buildings two methods are in use.

The first method is to make the drawing so complete that templets can be made for each individual piece, separately on the bench.

The second method is to give on the drawings only sufficient dimensions to locate the interior of the members and the position of the

pieces, leaving the templet-maker to work out the details on the laying-out floor.

The first method is illustrated in Fig. 95 and the second in Fig. 96. In the second method sufficient figures should be given to properly locate the main points in the truss; the interior pieces should be located by center-lines corresponding to the gage lines of the rivets, the centers of gravity lines or the outside edges of the pieces, as the case may be. The drawings should always indicate the number of rivets to be used in each connection, the size of rivets, the usual rivet pitch, and the minimum pitch allowed.

Erection Plan.—The erection plan should be made very complete. All the notes that it is necessary for the erecter to have, should be put on the erection plans; how much of the structure is to be riveted and how much bolted, whether it is to be painted after erection or not, whether the windows and doors are to be erected or not, etc. Center line drawings are usually sufficient for the erection plans. The name and the size of the piece should be given and every piece should have a name.

The following method was used by the Gillette-Herzog Mfg. Co., for mill buildings, and was very satisfactory:

If the points of the compass are known, mark all pieces on the north side with the letter "N", those on the south side with the letter "S", etc. Mark girts N. G. 1; N. G. 2; etc. Mark all posts with a different number, thus: N. P. 1; N. P. 2; etc. Mark small pieces which are alike with the same mark; this would usually include everything except posts, trusses and girders, but in order to follow the general marking scheme, where pieces are alike on both sides of a building, change the general letter; e. g. N. G. 7 would be a girt on the north side and S. G. 7 the same girt on south side. Then in case the north and south sides are alike, only an elevation of one side need be shown, and under it a note thus: "Pieces on south side of building, in corresponding positions have the same number as on this side, but prefixed by the letter "S" instead of the letter "N." Mark trusses T. 1; T. 2; etc. Mark roof pieces R. 1; R. 2; etc.

The above scheme will necessarily have to be modified more or less according to circumstances; for example, where a building has different sections or divisions applying on the same order number, in which case each section or division should have a distinguishing letter which should prefix the mark of every piece. In such cases it will perhaps be well to omit other letters, such as N., S., etc., so that the mark will not be too long for easy marking on the piece. In general, however, the scheme should be followed of marking all the large pieces, whether alike or not, with a different mark. This would refer to pieces which are liable to be hauled immediately to their places from the cars. But for all smaller pieces which are alike, give the same mark.

For architectural buildings adopt the following general scheme of marking: The basement "A"; first floor "B"; second floor "C"; then mark all the pieces on the first floor B. 1; B. 2; etc.; columns between first and second floors B. C. 1; B. C. 2; etc.

It will greatly aid the detailing, checking and erection if small sections are made showing the principal connections, such as girt connections, purlin connections, etc.

The erection plans of a mill building drawn in accordance with these rules are shown in Fig. 167 and Fig. 168.

CHOICE OF SECTIONS.—In designing, it will be found economical to use minimum weights of sections, and to use sections that can be most easily obtained. As small a number of sizes should be used as is practicable where material is to be ordered from the mill, if good delivery is to be expected. The ease with which any section can be obtained in a mill order, depends upon the call that that particular mill is having for the given section. If there is a large demand for the section, it will be rolled at frequent intervals, while if there is little or no demand for the section, the rollings are very infrequent and a small order may have to wait for a long time before enough orders for the section will accumulate that will warrant a special rolling. The ease with which sections can be obtained will, therefore, depend upon the mill and the conditions of the market. The standard and permissible

sizes of sections in use by the American Bridge Company, are given in the following table.

Standard Angles.				Permissible Angles.	
6″ × 6″	6″ × 4″	8″ × 8″		6″ × 3½″	
4″ × 4″	5″ × 3½″	5″ × 5″		4″ × 3½″	
3½″ × 3½″	4″ × 3″	2¼″ × 2¼″		3½″ × 2½″	
3″ × 3″	3½″ × 3″	2″ × 2″		3″ × 2″	
2½″ × 2½″	3″ × 2½″				
	2½″ × 2″				

Standard Channels.		Permissible Channels
15″	8″	9″
12″	6″	7″
10″		5″

Standard I Beams.		Permissible I Beams.
20″	10″	24″
18″	8″	9″
15″	6″	7″
12″		5″

Permissible Tees.

3″ × 3″ × ⅜″ 2″ × 2″ × 5-16″

Permissible Zee Bars.

6″ 5″ 4″ 3″

Standard Flats.			
1½″	3″	6″	12″
1¾″	3½″	7″	14″
2″	4″	8″	
2¼″	4½″	9″	
2½″	5″	10″	

Standard Rounds.

⅝″ ¾″ ⅞″ 1″ 1¼″ 1½″

Standard Squares.

¾″ ⅞″ 1″ 1¼″ 1½″

Other sizes than those specified may be obtained, but the time of delivery will be very uncertain unless the order is large enough to warrant a rolling.

Deck beams, bulb angles and special section Z-bars are hard to get unless ordered in large quantities. Flats ½″ thick and under are very hard to get.

Flats under 4″ should be ordered by ½″ variation in width; flats and universal plates over 4″ should be ordered by 1″ variation in width.

CHAPTER XXVII.

Paints and Painting.

Corrosion of Steel.—If iron or steel is left exposed to the atmosphere it unites with oxygen and water to form rust. Where the metal is further exposed to the action of corrosive gases the rate of rusting is accelerated, but the action is similar to that of ordinary rusting. Rust is a hydrated oxide of iron, and forms a porous coating on the surface of the metal that acts as a carrier of oxygen and moisture, thus promoting the action of corrosion. If nothing is done to prevent or retard the corrosion of the iron and steel used in metal structures, the metal rapidly rusts away and the structure is short lived. Wrought iron is affected by corrosion more than cast iron, and steel is affected more than wrought iron.

The corrosion of iron and steel may be prevented or retarded by covering it with a coating that is not affected by the corroding agents. This is very effectually accomplished by galvanizing; but on account of the cost it is impracticable to use the process for coating anything but sheet steel and small pieces of structural steel. The most common methods of protecting iron and steel are by means of a coating of paint, or by imbedding it in concrete.

PAINT.—The paints in use for protecting structural steel may be divided into oil paints, tar paints, asphalt paints, varnishes, lacquers, and enamel paints. The last two mentioned are too expensive for use on a large scale and will not be considered.

OIL PAINTS.—An oil paint consists of a drying oil or varnish and a pigment, thoroughly mixed together to form a workable mixture. "A good paint is one that is readily applied, has good covering powers,

adheres well to the metal, and is durable." The pigment should be inert to the metal to which it is applied and also to the oil with which it is mixed. Linseed oil is commonly used as the varnish or vehicle in oil paints, and is unsurpassed in durability by any other drying oil. Pure linseed oil will, when applied to a metal surface, form a transparent coating that offers considerable protection for a time, but is soon destroyed by abrasion and the action of the elements. To make the coating thicker, harder and more dense, a pigment is added to the oil. An oil paint is analogous to concrete, the linseed oil and pigment in the paint corresponding to the cement and the aggregate in the concrete. The pigments used in making oil paints for protecting metal may be divided into four groups as follows: (1) lead; (2) zinc; (3) iron; (4) carbon.

Linseed Oil.—Linseed oil is made by crushing and pressing flaxseed. The oil contains some vegetable impurities when made, and should be allowed to stand for two or three months to purify and settle before being used. In this form the oil is known as raw linseed oil, and is ready for use. Raw linseed oil dries (oxidizes) very slowly and for that reason is not often used in a pure state for structural iron paint. The rate of drying of raw linseed oil increases with age; an old oil being very much better for paint than that which has been but recently extracted. Raw linseed oil can be made to dry more rapidly by the addition of a drier or by boiling. Linseed oil dries by oxidation and not by evaporation, and therefore any material that will make it take up oxygen more rapidly is a drier. A common method of making a drier for linseed oil is to put the linseed oil in a kettle, heat it to a temperature of 400 to 500 degrees Fahr., and stir in about four pounds of red lead or litharge, or a mixture of the two, to each gallon of oil. This mixture is then thinned down by adding enough linseed oil to make four gallons for each gallon of raw oil first put in the kettle. The addition of four gallons of this drier to forty gallons of raw oil will reduce the time of drying from about five days to twenty-four hours. A drier made in this way costs more than the pure linseed oil, so that driers are very often made by mixing lead or manganese oxide with

rosin and turpentine, benzine, or rosin oil. These driers can be made for very much less than the price of good linseed oil, and are used as adulterants; the more of the drier that is put into the paint, the quicker it will dry and the poorer it becomes. Japan drier is often used with raw oil, and when this or any other drier is added to raw oil in barrels, the oil is said to be "boiled through the bung hole."

Boiled linseed oil is made by heating raw oil, to which a quantity of red lead, litharge, sugar of lead, etc., has been added, to a temperature of 400 to 500 degrees Fahr., or by passing a current of heated air through the oil. Heating linseed oil to a temperature at which merely a few bubbles rise to the surface makes it dry more rapidly than the unheated oil; however, if the boiling is continued for more than a few hours the rate of drying is decreased by the boiling. Boiled linseed oil is darker in color than raw oil, and is much used for outside paints. It should dry in from 12 to 24 hours when spread out in a thin film on glass. Raw oil makes a stronger and better film than boiled oil, but it dries so slowly that it is seldom used for outside work without the addition of a drier.

Lead.—*White Lead* (hydrated carbonate of lead—specific gravity 6.4) is used for interior and exterior wood work. White lead forms an excellent pigment on account of its high adhesion and covering power, but it is easily darkened by exposure to corrosive gases and rapidly disintegrates under these conditions, requiring frequent renewal. It does not make a good bottom coat for other paints, and if it is to be used at all for metal work it should be used over another paint.

Red Lead (minium; lead tetroxide—specific gravity 8.3) is a heavy, red powder approximating in shade to orange; is affected by acids, but when used as a paint is very stable in light and under exposure to the weather. Red lead is seldom adulterated, about the only substance used for the purpose being red oxide. Red lead is prepared by changing metallic lead into monoxide litharge, and converting this product into minium in calcining ovens. Red lead intended for paints must be free from metallic lead. One ounce of lampblack added to one pound of red lead changes the color to a deep chocolate and increases the

time of drying. This compound when mixed in a thick paste will keep 30 days without hardening.

Zinc.—Zinc white (zinc oxide—specific gravity 5.3) is a white loose powder, devoid of smell or taste and has a good covering power. Zinc paint has a tendency to peel, and when exposed there is a tendency to form a zinc soap with the oil which is easily washed off, and it therefore does not make a good paint. However, when mixed with red oxide of lead in the proportions of 1 lead to 3 zinc, or 2 lead to 1 zinc, and ground with linseed oil, it makes a very durable paint for metal surfaces. This paint dries very slowly, the zinc acting to delay hardening about the same as lampblack.

Iron Oxide.—Iron oxide (specific gravity 5) is composed of anhydrous sesquioxide (hematite) and hydrated sesquioxide of iron (iron rust). The anhydrous oxide is the characteristic ingredient of this pigment and very little of the hydrated oxide should be present. Hydrated sesquioxide of iron is simply iron rust, and it probably acts as a carrier of oxygen and accelerates corrosion when it is present in considerable quantities. Mixed with the iron ore are various other ingredients, such as clay, ocher and earthy materials, which often form 50 to 75 per cent of the mass. Brown and dark red colors indicate the anhydrous oxide and are considered the best. Bright red, bright purple and maroon tints are characteristic of hydrated oxide and make less durable paints than the darker tints. Care should be used in buying iron oxide to see that it is finely ground and is free from clay and ocher.

Carbon.—The most common forms of carbon in use for paints are lampblack and graphite. Lampblack (specific gravity 2.6) is a great absorbent of linseed oil and makes an excellent pigment. Graphite (blacklead or plumbago—specific gravity 2.4) is a more or less impure form of carbon, and when pure is not affected by acids. Graphite does not absorb nor act chemically on linseed oil, so that the varnish simply holds the particles of pigment together in the same manner as the cement in a concrete. There are two kinds of graphite in common use for paints—the granular and the flake graphite. The Dixon Graphite Co., of Jersey City, uses a flake graphite combined with silica,

while the Detroit Graphite Manufacturing Co., uses a mineral ore with a large percentage of graphitic carbon in granulated form. On account of the small specific gravity of the pigment, carbon and graphite paints have a very large covering capacity. The thickness of the coat is, however, correspondingly reduced. Boiled linseed oil should always be used with carbon pigments.

Mixing the Paint.—The pigment should be finely ground and should preferably be ground with the oil. The materials should be bought from reliable dealers, and should be mixed as wanted. If it is not possible to grind the paint, better results will usually be obtained from hand mixed paints made of first class materials than from the ordinary run of prepared paints that are supposed to have been ground. Many ready mixed paints are sold for less than the price of linseed oil, which makes it evident that little if any oil has been used in the paint. The paint should be thinned with oil, or if necessary a small amount of turpentine may be added; however turpentine is an adulterant and should be used sparingly. Benzine, gasoline, etc., should never be used in paints, as the paint dries without oxidizing and then rubs off like chalk.

Proportions.—The proper proportions of pigment and oil required to make a good paint varies with the different pigments, and the methods of preparing the paint; the heavier and the more finely ground pigments require less oil than the lighter or coarsely ground while ground paints require less oil than ordinary mixed paints. A common rule for mixing paints ground in oil is to mix with each gallon of linseed oil, dry pigment equal to three to four times the specific gravity of the pigment, the weight of the pigment being given in pounds. This rule gives the following weights of pigment per gallon of linseed oil: white lead, 19 to 26 lbs.; red lead, 25 to 33 lbs.; zinc, 15 to 21 lbs.; iron oxide, 15 to 20 lbs.; lampblack, 8 to 10 lbs.; graphite, 8 to 10 lbs. The weights of pigment used per gallon of oil varies about as follows: red lead, 20 to 33 lbs.; iron oxide, 8 to 25 lbs.; graphite, 3 to 12 lbs.

Covering Capacity.—The covering capacity of a paint depends upon the uniformity and thickness of the coating; the thinner the coat-

ing the larger the surface covered per unit of paint. To obtain any given thickness of paint therefore requires practically the same amount of paint whatever its pigment may be. The claims often urged in favor of a particular paint that it has a large covering capacity may mean nothing but that an excess of oil has been used in its fabrication. An idea of the relative amounts of oil and pigment required, and the covering capacity of different paints may be obtained from the following table.

AVERAGE SURFACE COVERED PER GALLON OF PAINT.*

Paint.	Volume of oil.	Lbs. of Pigment.	Volume and Weight of Paint.	Square Feet.	
				1 Coat.	2 Coats.
			Gals. Lbs.		
Iron Oxide (powdered)	1 gal	8.00	1.2=16.00	600	350
" " (ground in oil)..	1 "	24.75	2.6=32.75	630	375
Red Lead (powdered)..	1 "	22.40	1.4=30.40	630	375
White Lead (g'rd in oil).	1 "	25.00	1.7=33.00	500	300
Graphite (ground in oil).	1 "	12.50	2.0=20.50	630	350
Black Asphalt.....	1 "(turp.)	17.25	4.0=30.00	515	310
Linseed oil (no pigment)	1 "	....	.....	875	....

Light structural work will average about 250 square feet, and heavy structural work about 150 square feet of surface per net ton of metal.

It is the common practice to estimate ½ gallon of paint for the first coat and ⅜ gallon for the second coat per ton of structural steel, for average conditions.

Applying the Paint.—The paint should be thoroughly brushed out with a round brush to remove all the air. The paint should be mixed only as wanted, and should be kept well stirred. When it is necessary to apply paint in cold weather, it should be heated to a temperature of 130 to 150 degrees Fahr.; paint should not be put on in freezing weather. Paint should not be applied when the surface is damp, or during foggy weather. The first coat should be allowed to stand for three or four days, or until thoroughly dry, before applying

*Pencoyd Handbook, page 293.

the second coat. If the second coat is applied before the first coat has dried, the drying of the first coat will be very much retarded.

Cleaning the Surface.—Before applying the paint all scale, rust, dirt, grease and dead paint should be removed. The metal may be cleaned by pickling in an acid bath, by scraping and brushing with wire brushes, or by means of the sand blast. In the process of pickling the metal is dipped in an acid bath, which is followed by a bath of milk lime, and afterwards the metal is washed clean in hot water. The method is expensive and not satisfactory unless extreme care is used in removing all traces of the acid. Another objection to the process is that it leaves the metal wet and allows rusting to begin before the paint can be applied. The most common method of cleaning is by scraping with wire brushes and chisels. This method is slow and laborious. The method of cleaning by means of a sand blast has been used to a limited extent and promises much for the future. The average cost of cleaning five bridges in Columbus, Ohio, in 1902, was 3 cts. per square foot of surface cleaned.* The bridges were old and some were badly rusted. The painters followed the sand blast and covered the newly cleaned surface with paint before the rust had time to form.

Mr. Lilly estimates the cost of cleaning light bridge work at the shop with the sand blast at $1.75 per ton, and the cost of heavy bridge work at $1.00 per ton. In order to remove the mill scale it has been recommended that rusting be allowed to start before the sand blast is used. One of the advantages of the sand blast is that it leaves the surface perfectly dry, so that the paint can be applied before any rust has formed.

Cost of Paint.—The following costs of paints will give an idea of costs and proportions used:**

Oxide of Iron (Prince's Metallic Brown). One gallon of paint.

6¼ lbs. mineral at 1 cent..... 6 cts.

6¼ lbs. raw linseed oil—5-6 gallon at 56 cents.....47 "

Cost of materials per gallon of paint.....53 cts.

*Sand Blast Cleaning of Structural Steel, by G. W. Lilly, Transactions A. Soc. C. E., Feb., 1903.

**Walter G. Berg, Engineering News, June 6, 1895.

Red Lead (National Paint Co.). One gallon of paint.

20 lbs. red lead at 5 cents	$1.00
5½ lbs. raw linseed oil—¾ gallon at 56 cents	.42
Cost of materials per gallon of paint	$1.42

Graphite Paint (Dixon's Graphite). Five pounds of graphite paste and 1 gallon of oil make 1½ gallons of paint.

3¾ lbs. graphite paste at 12 cents	45 cts.
¾ gallon boiled linseed oil at 59 cents	44 "
Cost of materials per gallon of paint	89 cts.

Mr. A. H. Sabin in a paper read before the American Society of Civil Engineers, June, 1895, gives the following as the minimum costs of paints: Iron Oxide paint, 6¼ lbs. of oxide worth 9½ cents; 6¼ lbs. of oil worth 46½ cents; mixing in a mill, barrels, etc., 5 cents; making the actual cost of the paint 60 cents per gallon. The cost of a gallon of pure lead paint using 20 lbs. of red lead per gallon and oil at 56 cents per gallon will cost not less than $1.50 per gallon.

Cost of Painting.—The cost of applying the paint depends upon the condition of the surface to be painted, and upon other conditions. A common rule for ordinary work is that the cost of painting is about two to three times the cost of a good quality of paint required for the job. The cost of labor may not be more than the cost of the paint, and may be four or five times as much. The cost of painting light structural work in which considerable climbing has to be done is very difficult to estimate. The average cost of painting four bridges in Denver, Col., with a finishing coat of Goheen's Carbonizing, in 1899, was 51 cents for paint and 80 cents for labor, per ton of metal painted.

Priming Coat.—Engineers are very much divided as to what makes the best priming coat; some specify a first coat of pure linseed oil and others a priming coat of paint. Linseed oil makes a transparent coating that allows imperfections in the workmanship and rusted spots to be easily seen; it is not permanent however, and if the metal is exposed for a long time the oil will often be entirely removed before the second coat is applied. It is also claimed that the paint will not adhere

oil gives better results if applied hot to the metal. Another advantage of using oil as a priming coat is that the erection marks can be painted over with the oil without fear of covering them up. Red lead paint toned down with lampblack is probably used more for a priming coat than any other paint; the B. & O. R. R., uses 10 ozs. of lampblack to every 12 lbs. of red lead.

Without going further into the controversy it would seem that there is very little choice between linseed oil and a good red lead paint for a priming coat.

Finishing Coat.—From a careful study of the question of paints, it would seem that for ordinary conditions, the quality of the materials and workmanship is of more importance in painting metal structures than the particular pigment used. If the priming coat has been properly applied there is no reason why any good grade of paint composed of pure linseed oil and a very finely ground, stable and chemically non-injurious pigment will not make a very satisfactory finishing coat. Where the paint is to be subjected to the action of corrosive gases or blasts, however, there is certainly quite a difference in the results obtained with the different pigments. The graphite and asphalt paints appear to withstand the corroding action of smelter and engine gases better than red lead or iron oxide paints; while red lead is probably better under these conditions than iron oxide. Portland cement paint is the only paint that will withstand the action of engine blasts, and its use is now entirely in the experimental stage.

Conclusion.—It is urged against red lead paint, that the oil and the lead form a lead soap which is unstable; against iron oxide paint, that since the paint contains more or less iron rust it is necessarily a promoter of rust; against graphite paint, that there is not enough body in the pigment to make a substantial paint; etc. There is more or less truth in all the accusations made against the different kinds of paint, if the paint be bought ready mixed, or if made out of poor materials; however, with a good pigment and pure linseed oil, none of the above objections are of weight.

all dirt, grease, etc., and apply the finishing coats—preferably not less than two coats—giving ample time for each coat to dry before applying the next. Painting should not be done in rainy weather, or when the metal is damp, nor in cold weather unless special precautions are taken to warm the paint. The best results will usually be obtained if the materials are purchased in bulk from a responsible dealer and the paint ground as wanted. Good results are obtained with many of the patent or ready mixed paints, but it is not possible in this place to go into a discussion of their respective merits.

ASPHALT PAINT.—Many prepared paints are sold under the name of asphalt that are mixtures of coal tar, or mineral asphalt alone, or combined with a metallic base, or oils. The exact compositions of the patent asphalt paints are hard to determine. Black bridge paint made by Edward Smith & Co., New York City, contains asphaltum, linseed oil, turpentine and Kauri gum. The paint has a varnish-like finish and makes a very satisfactory paint. The black shades of asphalt paint are the only ones that should be used.

COAL-TAR PAINT.—Coal-tar used for painting iron work should be purified from all constituents of an acid nature; for this reason it is preferable to employ coal-tar pitch and convert it into paint by solution in benzine or petroleum. Tar paint should preferably be applied while hot. Oil paint will not stick to tar, and when repainting a surface that has been painted with tar it is necessary to scrape the surface if a good job is desired. Tar paint does not become hard and will run in hot weather; it is therefore not a desirable paint to use for many purposes.

CEMENT AND CEMENT PAINT.—Experiments have shown that a thin coating of Portland cement is effective in preventing rust; that a concrete to be effective in preventing rust must be dense and made very wet. The steel must be clean when imbedded in the concrete. There is quite a difference of opinion as to whether the metal should be painted before being imbedded or not. It is probably best to paint the

metal if it is not to be imbedded at once, or is not to be used in concrete-steel construction where the adhesion of the cement to the metal is an essential element. When the metal is to be imbedded immediately it is better not to paint it.

Portland Cement Paint.—A Portland cement paint has been used on the High St. viaduct in Columbus, Ohio, with good results. The viaduct was exposed to the fumes and blasts from locomotives, so that an ordinary paint did not last more than six months even on the least exposed portions. The method of mixing and applying the paint is described in Engineering News, April 24th and June 5th, 1902, as follows: "The surface of the metal was thoroughly cleaned with wire brushes and files—the bridge had been cleaned with a sand blast the previous year. A thick coat of Japan drier was then applied and before it had time to dry a coating was applied as follows: Apply with a trowel to the minimum thickness of 1-16 inch and a maximum thickness of ¼ inch (in extreme cases ½ inch) a mixture of 32 lbs. Portland cement, 12 lbs. dry finely ground lead, 4 to 6 lbs. boiled linseed oil, 2 to 3 lbs. Japan drier." After a period of about two years the coating was in almost perfect condition and the metal under the coating was as clean as when painted. The cost of the coating including the hand cleaning, materials and labor was 8 cents per square foot.

While this method of protecting metal is somewhat expensive it will certainly pay for itself in many places around smelters and shops.

References on Paint and Painting.—For a more complete discussion of the subject of paints the reader should consult the following:

Iron Corrosion by Louis E. Andes.

The Painting and Sand Blast Cleaning of Steel Bridges and Viaducts, by George W. Lilly, Engineering News, April 24th, 1902.

Rustless Coatings of Iron and Steel, by M. P. Wood, Transactions American Society of Mechanical Engineers, Vols. 15 and 16.

Preservation of Iron Structures Exposed to the Weather, by E. Gerber, Transactions American Society of Civil Engineers, May, 1895.

Painting Iron Railway Bridges, by Walter G. Berg, Engineering News, June 6, 1895.

Paints and Varnishes, by A. H. Sabin, Association of Engineering Societies, February, 1900.

Application of Paints, Varnishes, and Enamels for the Protection of Iron and Steel Structures and Hydraulic Work—a pamphlet for free distribution by Edward Smith & Company, New York.

CHAPTER XXVIII.

Estimate of Weight and Cost.

ESTIMATE OF WEIGHT.—The contract drawings for mill buildings are usually general drawings about like those in Fig. 167 and Fig. 168, in which the main members and the outline of the building are shown, together with enough sketch details to enable the detailer to properly detail the work. In making an estimate of weight from general drawings it is necessary that the estimater be familiar with the style of the details in use at the shop, and with the per cent of the main members that it is necessary to add, to provide for details and get the total shipping weight of the structure. There are two methods of allowing for details: (1) to add the proper per cent for details to the weight of each main member in the structure, and (2) to add a per cent for details to the total weight of the main members in the structure. The first method is the safest one to follow, although the second gives good results when used by an experienced man. The best way to obtain data on the per cents of details of different members in buildings and other structures is to make detailed estimates from the shop drawings. By checking these data with the actual shipping weights, the engineer will soon have information that will be invaluable to him. Second hand data on estimating are of comparatively little value for the reason that the conditions under which they hold good are rarely noted, and it is better that the novice work out his own data and depend on his own resources, at least until he has developed his estimating sense. In short the only way to learn to estimate, is to estimate.

The method of making estimates will be illustrated by making an estimate from the working drawings of a steel transformer building, the general plans of which are shown in Figs. 167 and 168. The

drawing, and the "Details" are those members whose sizes are supplied by the detailer. The building is a steel frame building, 60′ 0″ wide, 80′ 0″ long, 20′ 0″ posts, pitch of roof ⅓, and is covered with corrugated

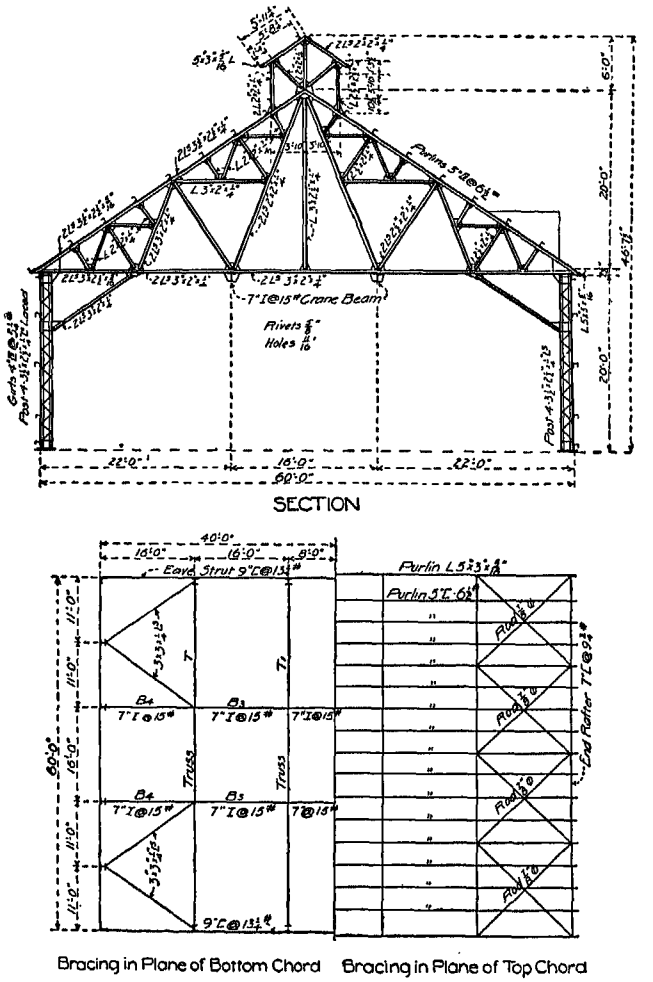


FIG. 167. CROSS-SECTION AND PLAN OF STEEL TRANSFORMER BUILDING.

steel. The general plans of the framework are shown in Figs. 167 and 168, and the plans and details of the corrugated steel are shown in Figs. 128 and 129.

The weights of the different sections were obtained from Cambria Steel. The estimate is self explanatory.

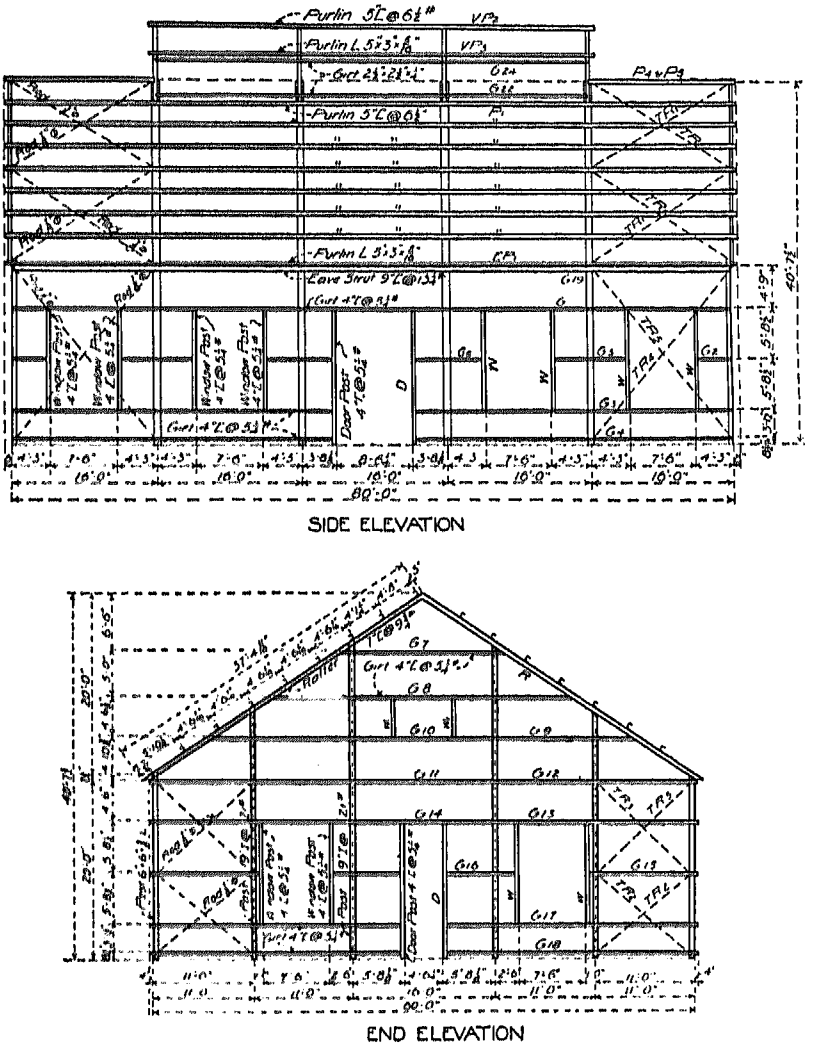


FIG. 168. SIDE AND END ELEVATIONS OF STEEL TRANSFORMER BUILDING.

ESTIMATE OF WEIGHT

Steel Frame Transformer Building, 60'-0" Wide, 80'-0" Long, 20'-0" Posts. Pitch of Roof ⅓. Covered with Corrugated Iron.

Number of Pieces	Shape	Section	Length Feet	Length Inches	Weight per Foot	Weight Main Members	Weight Details	Details Per Cent of Main Members	Total Weight
4 Trusses each thus:-									
4	L8	3½ × 2½ × 5/16	19	5 11/32	6.0	463			
4	L8	3½ × 2½ × ¼	18	0 5/32	4.9	353			
4	L8	3 × 2 × 5/16	11	1	5.0	222			
4	L8	3 × 2 × ¼	10	10	4.0	173			
2	L8	3 × 2 × ¼	16	2	4.0	259			
8	L6	2 × 2 × ¼	2	9	3.2	70			
4	L8	2½ × 2 × ¼	5	10	3.6	84			
8	L8	2 × 2 × ¼	5	1	3.2	130			
4	L8	3 × 2 × ¼	10	5½	4.0	167			
4	L8	2½ × 2 × ¼	11	10	3.6	170			
8	L8	2 × 2 × ¼	10	9	3.2	271			
1	L	3 × 2 × ¼	19	2	4.0	77			
4	L8	2 × 2 × ¼	6	0	3.2	77			
6	L8	2 × 2 × ¼	5	7¾	3.2	90			
2	L8	2 × 2 × ¼	4	7¾	3.2	29			
4	L8	3 × 2 × ¼	11	7¼	3.2	186			
2	Pls.	18 × ⅝	2	4	9.13		89		
2	Pls	7 × ½		7½	5.95		7		
2	Pls	4½ × ¼		9¾	3.01		6		
6	Pls	7 × ¼	1	0	2.95		36		
11	Pls	7½ × ¼		8¼	6.16		42		
4	Pls	8½ × ¼	1	0	7.22		29		
2	Pls	8½ × ¼	1	2	7.22		17		
4	Pls	12 × ¼	1	10	10.2		73		
2	Pls	16½ × ¼	1	8	13.81		40		
1	Pls	24 × ½	3	2	20.4		65		
4	Pls	2 × ⅝	1	5½	2.55		15		
12	Pls	6½ × ¼		7	5.55		47		
2	Pls	9⅝ × ¼		7	8.18		9		
2	Pls	5½ × ¼	1	0	4.46		8		
2	L8	2½ × 2½ × ½		9¾	4.0		8		
16	L8	3½ × 2½ × ½		7	4.9		45		
2	L8	3½ × 2½ × ⅝		3⅛	4.9		3		
1348		⅝ Rivet Heads	Per	100	9.95		134		
36		⅝ Washers	Per	100	3.00				
						2821	684	24.6	
		Total Weight of 4 Trusses = 3505 × 4 =							14020
8 Posts each thus:-									
4	L8	3½ × 2½ × ¼	19	11⅛	4.9	381			
1	Pl	10¼ × ½	2	0	8.71		16		
4	L8	3½ × 2½ × ¼		7½	5.9		15		
1	Pl	13 × ½	1	2½	11.05		13		
1	Pl	10 × ⅝	1	11	21.25		41		
2	L8	6 × 3½ × ⅝		10	11.6		19		
2	L8	6 × 4 × ⅝		9½	12.3		21		
17	Bars	2 × ½	1	7½	1.7		47		
160		⅝ Rivet Heads	Per	100	9.95		16		
						391	188	48.1	
		Total Weight of 8 Posts = 579 × 8 =							4632
									18652

Number of Pieces	Shape	Section	Length Feet	Length Inches	Weight per Foot	Weight Main Member	Weight Details	Details in Per Cent of Main Members	Total Weight
4	Posts, each thus:-						Brought Forward 18652		
1	I	9" @ 21#	35'	2¾	21	740			
8	L	3½ x 3½ x 5/16		5½	7.1		26		
2	L	6 x 4 x ⅜		6½	12.3		13		
2	L	6 x 3½ x ⅜		9	11.6		17		
1	Pl	9 x ⅜ x ⅜	1	5	19.13		27		
88		⅞" Rivet Heads	per 100		9.95		9		
		Total Weight 4 Posts =			837 x 4	740 =	92	12.5	3328
4	Posts, each thus:-								
1	I	9" @ 21#	27	10¾	21	586			
6	L	3½ x 3½ x 5/16		5½	7.1		20		
1	L	6 x 6 x ⅜		9	14.8		11		
2	L	6 x 4 x ⅜		6½	12.3		13		
2	L	6 x 3½ x ⅜		9	11.6		17		
1	Pl	9 x ⅜	1	5	19.13		27		
80		⅞" Rivet Heads	per 100		9.95		8		
		Total Weight 4 Posts =			682 x 4	586 =	96	16.4	2728
4	Posts, each thus:-								
1	L	6 x 6 x ⅜	20	8¾	14.8	306			
10	Pls	7 x ¼		6½	3.95		33		
1	Pl	5 x ¼		9½	4.25		3		
1	Pl	10 x ⅜		10	21.25		18		
1	L	5 x 3½ x 5/16	1	1	8.7		9		
3	L	5 x 3½ x 5/16		8	8.7		17		
1	L	3½ x 3½ x ¼		3	7.1		3		
2	L	6 x 3½ x ⅜		9½	11.6		18		
100		⅞" Rivet Heads	per 100		9.95		10		
		Total Weight 4 Posts =			417 x 4	306 =	111	36.3	1668
4	End Rafters, each thus:-								
1	C	7 @ 9¾#	37	5⅜	9.75	365			
		Con L and Pls					79		
		Total Weight 4 End Rafters =			444 x 4	365 =	79	22.0	1776
2	Eave Struts, each thus:-								
5	B	9 @ 13¼	16	0	13.25	1060			
		Total Weight 2 Eave Struts =			1060 x 2	=			2120
	Bottom Chord Bracing:-								
8	L	3 x 3 x ¼	18	0	4.9	706			
10	I	7" @ 15#	15	2	15.0	2275			
36	L	6 x 4 x ⅜		3	12.3		184		
4	L	3 x 3 x ¼		5	4.9		8		
128		⅞" Rivet Heads	per 100		9.95		13		
		Total Weight Bottom Chord Bracing =				2981	205	7.0	3186
	Purlins:-								
30	B	5 @ 6½#	32	7¾	6.5	6355			
14	B	"	15	11½	6.5	1452			
6	B	"	16	7¾	6.5	650			
6	L	5 x 3 x 5/16	32	7¾	8.2	1604			
2	L	"	15	11½	8.2	262			
2	L	"	16	7¾	8.2	272			10595
		Total Weight of Purlins =							44053

Number of Pieces	Shape	Section	Length Feet	Length Inches	Weight Per Foot	Weight Main Members	Weight Details	Details in Percent of Main Members	Total Weight
							Brought Forward		44053
Girts:—									
	Cs	4" @ 5¼#	1664	lin ft.	5.25	8736			
	Ls	3"x3"x¼"	96	" "	4.9	470			
	Ls	2½"x2½"x¼"	96	" "	4.0	384			
			Total Weight						9590
Rods.—									
8	Rods	⅞" ϕ	28	0	2.0	448			
16	Rods	⅞" ϕ	18	0	2.0	576			
16	Rods	⅞" ϕ	23	0	2.0	736			
32	Bolts	Anchor ⅞" ϕ	1	6	2.0		96		
4	"	" "	1	0	2.0		8		
18	Rods	Louvre ⅝" ϕ	3	0	1.0		54		
18	Bolts	U ⅝" ϕ			1.5		9		
18		Spring Catches			0.5		9		
36	Pls	Anchor 4"x 5/16"		4	1.4		50		
56	Pins	Cotter 1⅜"x2⅜"			1.3		74		
				Total Weight		1760	300	17.0	2060
			Total Weight			Steel Framework			55703
Corrugated Iron.—									
84	Squares No 22 as per list, per sq.				138	11592			
70	" 24 " " " "				111	7770			
	Ridge Roll	. 22 Black					250		
	Flashing	. 22 "					414		
	Cornice	. 20					1500		
	Louvres	. 20 and No16					2100		
						19362	4264		
			Total Weight			Corrugated Iron			23626
			Total Weight			Steel			79329
Summary —									
	Trusses					11284	2736	24.6	14020
	4L Posts					3128	1504	48.1	4632
	IB "					5304	752	14.1	6056
	L "					1224	444	36.3	1668
	End Rafters					1460	316	22.0	1776
	Eave Struts					2120		0 0	2120
	Bottom Chord Bracing					2981	205	7.0	3186
	Rods					1760	300	17.0	2060
	Weight Excluding Purlins and Girts					29261	6257	22 0	35518
	Purlins					10595			10595
	Girts					9590			9590
	Total Frame work					49446	6257	13.0	55703
	Corrugated Iron								23626
	Total Weight,								79329

The weights and per cents of three other buildings, are shown in Table XXVI. The estimates for these buildings were made from the shop drawings, and were checked with the shipping weights. These buildings are of light construction with end post bents, (a) Fig. 1.

ESTIMATE OF COST

Classification of Material	Cost of Material			Cost of Labor	
	Weight	Price	Amount	Price	Amount
Riveted Trusses	14020	¢1.60	$224.32	¢1.00	$140.20
Latticed Columns	4632	1.60	74.11	1.00	46.32
I Beam "	6056	1.65	99.92	.50	30.28
L "	1668	1.60	26.69	.50	8.34
C Struts	3896	1.60	62.34	.26	9.74
I Beam Bracing	2480	1.65	40.92	.25	6.20
L Bracing	706	1.60	11.30	.25	1.77
C Purlins	10595	1.60	169.52	.15	15.90
C Girts	9590	1.60	153.44	.15	14.39
Rods	2060	1.80	37.08	1.00	20.60
Corrugated Iron No. 22	11592	2.60	301.39		
" " " 24	7770	2.70	209.79		
Ridge Roll, Louvres, Etc.	4264	2.50	106.60	1.00	42.64
Asbestos Mill Board	1760	2.50	44.00		
Poultry Netting	400		32.50		
8-d Barbed Roofing Nails	32	3.00	.96		
40-d Wire Nails	5	2.50	.13		
540 Stove Bolts - $\frac{3}{8}$" x 1" (per 100)	8	36.00	1.94		
540 Washers 1" x $\frac{3}{8}$" x 4"	76	6.00	4.56		
540 Cut Washers ‡"	2	7.00	.14		
800 Wire Staples	5	4.00	.20		
Copper Rivets #8 - $\frac{3}{8}$" long	6	25.00	1.50		
1200 Carriage Bolts $\frac{1}{2}$" x 21" (per 100)	210	$1.10	13.20		
200 " $\frac{1}{2}$" x 3$\frac{1}{2}$" "	46	$1.50	3.00		
260 Wood Screws #14 - $\frac{3}{8}$" "	10	#.60	1.56		
54 Steel Butts 3$\frac{1}{2}$" x 3$\frac{1}{2}$"	60	¢8.00	4.32		
4 Mortise Door Locks	10	75.00	3.00		
18 10" T Hinges	40	12.00	2.16		
2 - 10" Foot Bolts	5	20.00	1.00		
2 - Chain Bolts	5	20.00	1.00		
96 Window Weights	1440	2.00	28.80		
24 " Locks	16	15.00	3.60		
24 " Lifts	10	10.00	2.40		
675 Lin. Ft. Sash Cord	30	15.00	4.50		
2-8 Light Windows & Frame	80	#4.00	8.00		
12-24 " " "	1600	$6.00	96.00		
2 Doors	800	#15.00	30.00		
2 Doors	200	#4.00	8.00		
Total Weight 87212		Cost Mat'l	$1813.89	Cost Labor	$336.38

SUMMARY		
Cost of Material		$1813.89
Cost of Shop Labor		336.38
Cost of Details	30 tons @ $3.60	108.00
Cost of Shop Painting		40.00
Total Shop Cost		2298.27
Freight, Mill to Shop	30 tons @ $5.00	150.00
Freight, Shop to Site	44 " @ $16.00	704.00
Erection, Structural	30 tons @ $8.00	240.00
" Corrugated	60 sqs. Roofing @ $1.25	75.00
	60 " Siding @ $.75	45.00
" Miscellaneous		50.00
Painting { 76 gals. Paint		76.00
Painting { Labor		75.00
Total Cost		$3813.27

TABLE XXVI.

WEIGHTS AND PER CENTS OF DETAILS OF MILL BUILDINGS.

Steel Mill Buildings with Self Supporting Frames covered on Roof and Sides with one thickness of Corrugated Iron.								
Part of Structure	40'-0"x40'-0"x14'-0" 2 Trusses 40'-0" Pitch ¼ End Framing Cor. Iron {Roof #24 Sides #24 2 Circular Ventilator		40'-0"x48'-0"x14'-0" 2 Trusses 40'-0" Pitch ¼ End Framing Cor. Iron {Roof #24 Sides #24 2 Circular Ventilator		60'-0"x75'-0"x18'-0" 4 Trusses 60'-0" Pitch ⅓ End Framing Cor. Iron {Roof #22 Sides #24 3 Circular Ventilator		60'-0"x80'-0"x20'-0" 4 Trusses 60'-0" Pitch ⅓ End Framing Cor. Iron {Roof #22 Sides #24 Monitor Ventilators	
	Weight	Details in per cent of Main Members	Weight	Details in per cent of Main Members	Weight	Details in per cent of Main Members	Weight	Details in per cent of Main Members
	lbs	per cent	lbs.	per cent	lbs.	per cent	lbs	per cent
Trusses	2848	25	2848	25	13940	34.4	14020	24.6
4 L Columns	1428	70	1428	70	3476	52.5	4632	48.1
I Beam	1148	15	1148	15	4251	33.0	6056	14.1
L Columns	912	36	952	36	1470	14.0	1668	36.3
End Rafters	1036	17	1076	22	3314	11.6	1776	20.0
Eave Struts	900	0	1080	0	3117	33.2	2120	0.0
Lower Chord Bracing	930	22	1049	20	2763	9.7	3186	7.0
Rods	900	15	920	17	1737	6.2	2060	12.0
Purlins	2281	5	3516	7	6713	4.7	10595	0.0
Girts	3170	2	3252	2	9895	10.0	9590	0.0
Weight of Framework	15553	19	17269	20	50676	24.0	55703	13.0
Weight per Sq. Ft	9.8		9.0		11.2		11.7	
Corrugated Iron	5880		6892		17000		23626	
Total of Steel	21433		24161		67676		79329	
Weight per Sq. Ft.	13.4		12.6		15.1		16.5	

Channel eave struts were used in all except the third building in which 4-angle laced struts were used. A very good idea of the per cent of details in the different parts of the structures can be obtained from Table XXVI. The details of riveted mill building trusses will commonly vary between the limits of 25 and 35 per cent as given in the table; being more often near 25 than 35 per cent. The per cents of details in trusses is practically independent of the length of span, and is larger for light than for heavy work. It should be noted that the details in columns is mostly due to the bases and connections—the per cents of details will therefore decrease as the length of the column increases. The weights of the other parts are so variable that no general rules can be given. Where a uniform per cent is added to the total weight of main members to provide for details, it is common to add about 30

per cent to the weight of the framework exclusive of the purlins and girts where end bent (b) Fig. 1, is used. In the estimate given it will be seen that the per cent is only 22, the small value being due in part to the use of channel eave struts and the end post framing.

In estimating the weight of corrugated steel add 25 per cent for laps where two corrugations side lap and 6 inches end lap are required, and 15 per cent where one corrugation side lap and 4 inches end lap are required.

The weights of the sections, rods, bolts, turnbuckles, etc., are obtained from Cambria Steel, or other handbook.

The engineer should use every care to check his work in making estimates, the material should be checked off the drawings, and the calculations should be carefully checked and rechecked. Slide rules and adding machines are invaluable in this work. No results should, however, be allowed to pass until they have been roughly checked by the engineer by aliquot parts, or by making a mental estimate of each quantity. The engineer can soon develop a sense of estimate, so to speak, and will often detect blunders intuitively. Accuracy is of more value in estimating than precision. While the method outlined may seem somewhat crude at first glance, it is nevertheless true that a preliminary estimate made by a skilled man will commonly be within 1 or 2 per cent of the shipping weight, and if off more than 2½ per cent it is pretty certain that there was something wrong either with the estimate or with the estimater. The estimated weight should be a little heavy rather than light, say 1 to 2 per cent.

ESTIMATE OF COST.—The cost of the different parts of a mill building varies with the local conditions, cost of labor, and cost of materials. The discussion of this subject will be divided into (1) cost of material, (2) cost of shop work, and (3) cost of erection. The cost of transportation must also be included in arriving at the total cost. The subject of costs is a very difficult one to handle and the author would caution the reader to use the data given on the following pages with care, for the reason that costs are always relative and what may

be a fair cost in one case may be sadly in error in another case which appears an exact parallel. The price of labor will be given in each case, or the costs will be based on a charge of 40 cents per hour which includes labor, cost of management, tools, etc.

Cost of Material.—The cost of structural steel can be obtained from the current numbers of the Iron Age, Engineering News, etc., or may be obtained direct from the manufacturers or dealers. In 1903 beams, channels, angles, plates, and bars were quoted at about 1.60 cents per pound f. o. b. Pittsburg. Beams 18, 20 and 24 inches deep take 0.10 cents per pound higher price than the base price for beams. The mills at present quote a delivered price only, equal to the mill price plus the usual freight charge. This price is often more than the customer could obtain by paying the freight himself, on account of the freight rebates that are often allowed.

Cost of Mill Details.—Mills are allowed a variation in length of sections of ¼ of an inch; which means that beams, channels, etc., may come ⅜ of an inch shorter or ⅜ of an inch longer than the length called for. When a less variation than this is required a special price is charged for cutting to exact length. The following list of mill extras adopted January, 1902, is now in force:

LIST OF EXTRAS TO BE ADDED TO PRICE OF PLAIN BEAMS AND CHANNELS.

1.	For cutting to length with less variation than plus or minus ⅜ inch.....	$0.15
2.	Plain punching one size hole in web only	.15
3.	Plain punching one size hole in one or both flanges	.15
4.	Plain punching one size hole in either web and one flange or web and both flanges	.25
5.	Plain punching each additional size hole in either web or flange, web and one flange or web and both flanges.....	.25
6.	Plain punching one size hole in flange and another size hole in web of the same beam or channel	.40
7.	Punching and assembling into girders....	.35

8.	Coping, ordinary beveling, including cutting to exact length, with or without punching, including the riveting or bolting of standard connection angles.....	.35
9.	For painting or oiling one coat with ordinary oil or paint.....	.10
10.	Cambering Beams and Channels and other shapes for ships or other purposes..	.25
11.	Bending or other unusual work.....	Shop rates
12.	For fittings, whether loose or attached, such as angle connections, bolts and separators, tie-rods, etc.....	1.55

The above prices are per 100 lbs. of steel.

In ordering material from the mill the following items should be borne in mind. Where beams butt at each end against some other member, order the beams ½ inch shorter than the figured lengths; this will allow a clearance of ¼ inch if all beams come ⅜ of an inch too long. Where beams are to be built into the wall, order them in full lengths making no allowance for clearance. Order small plates in multiple lengths. Irregular plates on which there will be considerable waste should be ordered cut to templet. Mills will not make reentrant cuts in plates. Allow ¼ of an inch for each milling for members that have to be faced. Order web plates for girders ¼ to ½ inch narrower than the distance back to back of angles. Order as nearly as possible every thing cut to required length, except where there is liable to be changes made, in which case order long lengths.

It is often possible to reduce the cost of mill details by having the mills do only part of the work, the rest being done in the field, or by sending out from the shop to be riveted on in the field connection angles and other small details that would cause the work to take a very much higher price. Standard connections should be used wherever possible, and special work should be avoided.

The classification of iron and steel bars is given in Table XXVII. The full extra charges for sizes other than those taking the base rate are seldom enforced; one-half card extras being very common.

TABLE XXVII.

Iron Classification.

Adopted Dec. 3, 1895, by National Bar Iron Association.
Adopted March 16, 1899, by Eastern Bar Iron Manufacturers' Association.

Rounds and Squares.

	Extra.
1/16	2 1/16
1/8	1 1/16
3/16	1/16
1/4 to 5/16	1/16
5/16 to 7/16	1/16
3/8 to 7/16	1/16
7/16 to 9/16	1/16
1/2 to 5/8	1/16
5/8 to 11/16	1/16
3/4 to 7/8	1/16

	Extra.
1 to 1 1/8	Base sizes no extra
2 to 2 1/8	1/16
3 to 3 1/2	1/16
3 3/8 to 4	1/16
4 1/8 to 4 1/2	1c.
4 3/8 to 5	1 1/16
5 1/8 to 6	1 1/16
6 1/8 to 6 1/2	2 1/16
6 3/8 to 7 1/4	2 1/16

Oval Iron.

	Extra.
3/8 to 7/16	1 1/16
1/2 to 5/8	1/16
1/2 to 9/16 x 1/8	1c.
5/8 to 11/16	1/16

	Extra.
5/8 to 11/16 x 3/8	1 1/16
3/4 to 1 1/8	1/16
7/8 to 1 1/2	1/16

Half Oval and Half Round.

	Extra.
1/4	4 1/16
5/16	3 1/16
3/8 to 7/16	2 1/16
1/2 to 5/8	1 1/16

	Extra.
5/8 to 11/16	1/16
3/4 to 1 1/8	1/16
7/8 to 2	1/16
2 1/4 to 3	1/16

Half ova's less than 1/4 their width in thickness, extra price.

Flats.

	Extra.
3/8 to 7/16 x 1/4 to 5/16	1 1/16
1/2 to 5/8 x 1/4 to 1/2	1c.
1/2 to 5/8 x 3/8 to 1/2	1/16
5/8 to 11/16 x 1/4 to 5/16	1/16
5/8 to 11/16 x 3/8 to 5/8	1/16
3/4 to 1 1/8 x 1/4 to 5/16	1/16
3/4 to 1 1/8 x 3/8 to 3/4	1/16
1 to 1 1/8 x 1/4 to 7/16	1/16
1 to 1 1/16 x 3/8 to 7/8	1/16
1 1/8 to 1 3/8 x 3/8 to 1	1/16
1 1/2 to 4 x 1/4 to 1/16	1/16
1 1/2 to 4 x 3/8 to 1	Base no extra.
1 3/8 to 4 x 1 1/16 to 1 1/2	1/16
2 to 4 x 1 3/8 to 2	1/16
2 to 4 x 2 1/8 to 3	1/16

	Extra.
4 1/4 to 6 x 1/4 to 5/16	1/16
4 1/4 to 6 x 3/8 to 1	1/16
4 1/4 to 6 x 1 1/16 to 1 1/2	1/16
4 1/4 to 6 x 1 3/8 to 2	1/16
4 1/4 to 6 x 2 1/8 to 3	1/16
6 1/4 to 6 3/4 x 1/4 to 5/16	1/16
7 to 8 x 1/4 to 5/16	1/16
6 1/4 to 8 x 3/8 to 1 1/2	1/16
6 1/4 to 8 x 1 3/8 to 2	1/16
6 1/4 to 8 x 2 1/8 to 3	1c.
8 1/4 to 10 x 1/4 to 5/16	1/16
8 1/4 to 10 x 3/8 to 1	1/16
8 1/4 to 10 x 1 1/16 to 1 1/2	1/16
8 1/4 to 10 x 1 3/8 to 2	1c.

Flats 7/8 thick 1/10c. per lb higher than 1/4 to 1/8 thick.
Bevel edge Shaft Iron 1/10c. higher than same size of Flats.
All round edge iron 1/10c. per lb extra.
Horse Shoe Iron all sizes 1c. extra.

Light Bands.

	Extra.
3/4 x Nos. 10, 11 and 12	1/16
5/8 x No. 9 to 1/16	1/16
7/16 to 1/2 x Nos. 10, 11 and 12	1/16
7/16 to 1/2 x No. 9 to 1/16	1/16
7/16 to 5/8 x Nos. 10, 11 and 12	1/16
7/16 to 5/8 x No. 9 to 1/16	1c.
11/16 to 3/4 x Nos. 10, 11 and 12	1/16
11/16 to 3/4 x No. 9 to 1/16	1/16
11/16 to 7/8 x Nos. 10, 11 and 12	1/16
11/16 to 7/8 x No. 9 to 1/16	1/16

	Extra.
1 to 1 1/16 x Nos. 10, 11 and 12	1/16
1 to 1 1/16 x No. 9 to 1/16	1/16
1 1/4 to 4 x Nos. 10, 11 and 12	1/16
1 1/4 to 4 x No. 9 to 1/16	1/16
4 1/4 to 6 x Nos. 10, 11 and 12	1/16
4 1/4 to 6 x Nos. 9 to 1/16	1/16
6 1/4 to 6 3/4 x Nos. 10, 11 and 12	1/16
6 1/4 to 6 3/4 x No. 9 to 1/16	1/16
7 to 8 x Nos. 10, 11 and 12	1c.
7 to 8 x No. 9 to 1/16	1/16

Bevel Edge Box Iron same as Light Bands of same sizes.
Beaded Band Iron 1 1/4 inch to 2 inch 1/10 extra.
Sand Band Iron 1/10c. above same sizes of Light Bands.

Shop Cost.—The shop cost of the various classes of work is a variable quantity, depending upon the equipment and capacity of the shop, the number of pieces made alike, the familiarity of the shop men with the particular class of work, and with the cost of labor. The costs given below are the average costs for a shop with a capacity of about 1000 tons per month that has made a specialty of mill building work. The costs given are based on a charge of 40 cents per hour for the number of hours actually consumed in getting out the contract. This charge is assumed to cover the cost of management, cost of operation and maintenance, as well as the cost of labor. The cost of management in a small shop is very small, but in a large concern it may amount to as much as 35 to 40 per cent of all the other charges combined. For this reason small structural shops can often fabricate light structural steel for a less cost than the large shops. The prices given are about an average of those used by the agents of the company above, and have been checked against actual costs for the greater part.

Columns.—In lots of at least six, the shop cost of columns is about as follows: Columns made of two channels and two plates, or two channels laced cost about 0.80 to 0.70 cents per lb., for columns weighing from 600 to 1000 lbs. each; columns made of 4-angles laced cost from 0.80 to 1.10 cents per lb.; columns made of two channels and one I beam, or three channels cost from 0.65 to 0.90 cents per lb.; columns made of single I beams, or single angles cost about 0.50 cents per lb.; and Z-bar columns cost from 0.70 to 0.90 cents per lb.

Plain cast columns cost from 1.50 to 0.75 cents per lb., for columns weighing from 500 to 2500 lbs., in lots of at least six.

Roof Trusses.—In lots of at least six, the shop cost of ordinary riveted roof trusses in which the ends of the members are cut off at right angles is about as follows: Trusses weighing 1000 lbs. each, 1.15 to 1.25 cents per lb.; trusses weighing 1500 lbs. each, 0.90 to 1.00 cents per lb.; trusses weighing 2500 lbs. each, 0.75 to 0.85 cents per lb.; and trusses weighing 3500 to 7500 lbs. 0.60 to 0.75 cents per lb. Pin connected trusses cost from 0.10 to 0.20 cents per lb. more than riveted trusses.

Eave Struts.—Ordinary eave struts made of 4-angles laced, whose length does not exceed 20 to 30 feet, cost for shop work from 0.80 to 1.00 cents per lb.

Plate Girders.—The shop work on plate girders for crane girders and floors will cost from 0.60 to 1.25 cents per lb., depending upon the weight, details and number made at one time.

Eye-Bars.—The shop cost of eye-bars varies with the size and length of the bars and the number made alike. The following costs are a fair average: Average shop cost of bars 3 inches and less in width and ¾ inches and less in thickness, is from 1.20 to 1.85 cents per lb., depending on length and size. A good order of bars running from 2½″ × ¾″ to 3″ × ¾″, and from 16 to 30 ft. long, with few variations in size, will cost about 1.20 cents per lb. Large bars in long lengths ordered in large quantities can be fabricated at from 0.55 to 0.75 cents per lb.

To get the total cost of eye-bars the cost of bar steel must be added to the shop cost.

Cost of Drafting.—The cost per ton for making details of mill buildings varies with the character of the work and the tonnage that is to be fabricated from one detail, so that costs per ton may mean very little. The following will give an idea of the range of costs. Details for headworks for mines cost from $4.00 to $6.00 per ton; details for church and court house roofs having hips and valleys cost from $6.00 to $8.00 per ton; details for ordinary mill buildings cost from $2.00 to $4.00 per ton. The details for all work fabricated by the Gillette-Herzog Mfg. Co., with the exception of plain beams and complicated tank work, were made in 1896 by contract, by Mr. H. A. Fitch, now structural engineer for the Minneapolis Steel and Machinery Co., Minneapolis, for $2.06 per ton. This price netted the contractor a fair profit.

Actual Costs of Detailing.—The details of the building for which the estimate is made in this chapter cost $3.60 per ton. The details for the Basin & Bay State Smelter, Basin, Montana, containing 270 tons of steel cost $2.00 per ton.

Actual Shop Costs.—The following actual shop costs will give an idea of the range of costs: The shop cost of the transformer building of which the estimate is made in this chapter was about $20.00 per ton including drafting; the Carbon Tipple building at Carbon, Montana, weighing 86 tons, cost $18.60 per ton. The shop cost of the structural work of the East Helena transformer building, the estimate of which is given in the next to the last column in Table XXVI, cost $21.80 per ton including details. The shop cost of the Basin & Bay State smelter, weighing 270 tons, was $17.20 per ton including details which cost $2.00 per ton. The shop cost of six gallows frames made by the Gillette-Herzog Mfg. Co., varied from $21.80 to $41.80 per ton, with an average of $32.20 per ton including details.

Cost of Erection.—With skilled labor at $3.50 and common labor at $2.00 per day of 9 hours, small buildings like those given in Table XXVI will cost about $10.00 per ton for the erection of the steel framework, if trusses are riveted and all other connections are bolted. The cost of laying corrugated steel is about $0.75 per square when laid on plank sheathing, $1.25 per square when laid directly on the purlins, and $2.00 per square when laid with anti-condensation roofing. The erection of corrugated steel siding costs from $0.75 to $1.00 per square. The cost of erecting heavy machine shops, all material riveted and including the cost of painting but not the cost of the paint, is about $8.50 to $9.00 per ton. Small buildings in which all connections are bolted may be erected for from $5.00 to $6.00 per ton. The cost of erecting the East Helena transformer building (next to the last building in Table XXVI) was $12.80 per ton including the erection of the corrugated steel and transportation of the men. The cost of erecting the Carbon Tipple was $8.80 per ton including corrugated steel. The cost of erection of the Basin & Bay State Smelter was $8.20 per ton including the hoppers and corrugated steel. The cost of erecting 6 gallows frames in Montana varied from $11.20 to $15.20 per ton, with an average of $13.00 per ton, all connections being riveted.

COST OF MISCELLANEOUS MATERIAL.—In making an estimate for a mill building the engineer needs to be familiar with the

costs of building hardware, lumber, etc. Prices of building hardware are usually quoted at a certain discount from standard lists. These standard lists and the discounts can be obtained from the dealers, or the method described in the following paragraph may be used.

The following method of obtaining costs of building hardware and other miscellaneous materials, has been found very satisfactory: Obtain standard lists from the dealers, or a very complete one entitled "The Iron Age Standard Hardware Lists" may be obtained from the David Williams Co., New York, for $1.00, postpaid. To find the current discount, consult the current number of the Iron Age, or a similar publication—the Iron Age is published by the David Williams Co., New York, at $5.00 per year, or 10 cents per single copy, and gives each week the current hardware prices and discounts. By applying the discount to the prices given in the standard lists the current price of the material can be obtained. The standard lists of machine bolts, carriage bolts, nails, and turnbuckles are given for convenience in estimating and to illustrate what is meant by lists.

To illustrate the method just described the current prices (1903) will be obtained for a few items:

2-in. Barbed Roofing Nails.—The base price of nails is $2.65 per keg of 100 lbs., and from standard nail list (Table XXVIII) we see that 2-in. barbed roofing nails take $0.35 per 100 lbs. advance over the base price making the price $3.00 per 100 lbs.

Carriage Bolts.—2½″ × ½″ carriage bolts are listed (Table XXVIII) at $3.00 per 100, from which a discount of 60 and 10% is allowed. A discount of 60 and 10% is equivalent to a discount of 64%, making the price of the bolts $1.08 per 100.

Machine Bolts.—4″ × ½″ machine bolts are listed (Table XXIX) at $4.90 per 100, from which a discount of 65 and 5% is allowed. A discount of 65 and 5% is equivalent to a discount of 66¾%, making the price of the bolts $1.36 per 100. The weight of machine bolts and nuts is given, from which the price per lb. can be obtained for any size of bolt.

The following approximate prices of materials will assist in making preliminary estimates: rivets, $2.25 to $3.00 per 100 lbs.; boat spikes, $2.25 to $3.00 per 100 lbs.; washers, cut, $6.00 per 100 lbs.; washers, cast, $1.50 to $2.00 per 100 lbs.; sash weights, $1.00 to $1.50 per 100 lbs.; pins $3.50 to $4.00 per 100 lbs.; pin nuts, $4.00 per 100 lbs.; wire poultry netting, $0.50 per square; asbestos felt, 32 lbs. to the square, $2.75 to $3.00 per 100 lbs.; turnbuckles, 50% discount. Other prices and costs will be found in the descriptions of the various articles on the preceding pages.

TABLE XXVIII.

LIST ADOPTED DECEMBER 1, 1896.

STEEL WIRE NAILS.

Common, Fence and Brads.	20d to 60d........	Base.
	10d to 16d........	$0.05 extra per keg.
	8d and 9d	.10 " "
	6d and 7d........	.20 " "
	4d and 5d........	.30 " "
	3d	.45 " "
	2d	.70 " "

Barbed Common and Barbed Car Nails, 15c. advance over Common.

Fine Nails........	3d	2d
(Extra per keg)........	$0.50	1 00

Casing and Smooth Box Nails—(Extra per keg).

30 & 40d	20d	12 & 16d	10d	8 & 9d	6 & 7d	5d	4d	3d	2d
$0.15	.15	.15	.15	.25	.35	.50	.50	.70	1.00

Finishing........	20d	12 & 16d	10d	8 & 9d	6 & 7d	5d	4d	3d	2d
(Extra per keg).	$0.25	.25	.25	.35	.45	.55	.65	.85	1.15

Barrel........	1¾ in.	1⅝	1½	1¼	1	⅞	¾
(Extra per keg).	$0 30	.40	.50	.60	.70	.85	1.00

Clinch	20d	12 & 16d	10d	8 & 9d	6 & 7d	4 & 5d	3d	2d
(Extra per keg).	$0 35	.35	.35	.45	.55	.65	.85	1.05

Barbed Roofing.	2 in.	1¾	1½	1¼	1	⅞	¾
(Extra per keg)	$0.35	.45	.45	.55	.60	.65	.75

Wire spikes, all sizes........ ....10c. extra.

LENGTH AND APPROXIMATE GAUGE OF COMMON WIRE NAILS.

2d	3d	4d	5d	6d	7d	8d	9d
1 in. No. 15	1¼ 14	1½ 12½	1¾ 12½	2 11½	2¼ 11½	2½ 10¼	2¾ 10¼

10d	12d	16d	20d	30d	40d	50d	60d
3 in. No. 9	3¼ 9	3½ 8	4 6	4½ 5	5 4	5½ 3	6 2

Turnbuckles.

Size. Inch.	Each.	Size. Inch.	Each.	Size. Inch.	Each.
⅜	$0.40	1⅛	$1.38	2⅛	$5.00
7/16	.42	1¼	1.50	2¼	5.50
½	.45	1⅜	1.75	2⅞	6.00
9/16	.48	1½	2.00	3	6.50
⅝	.50	1⅞	2.25	3⅛	7.50
¾	.63	2	2.65	3¼	8.00
⅞	.75	2⅛	3.10	3⅜	9.00
1	.88	2¼	3.50	3½	10.00
1⅛	1.00	2⅜	4.00	3¾	15.00
1¼	1.25	2½	4.50	4	20.00

Common Carriage Bolts.

List Feb'y 1, 1904
Price per 100.

Length in Inches.	1-4	5 16	3-8	7 16	1-2	9 16 & 5 8	3 4
1½	$1.00	$1.20	$1.00	$2.20	..	....	...
1¾	1.04	1.25	1.68	2.20	....	....	....
2	1.08	1.30	1.76	2.38	....	....	....
2¼	1.12	1 35	1.84	2 47			....
2½	1.16	1.40	1.92	2.56	$3 00	$5.20	$7.20
2¾	1.20	1.45	2 00	2.65	3 11	5.37	7 48
3	1.24	1 50	2.08	2.74	3.22	5.54	7.66
3¼	1 28	1.55	2.16	2 83	3 33	5.71	7.89
3½	1 32	1 60	2.24	2 92	3.44	5.88	8.12
3¾	1.36	1.65	2.32	3.01	3 55	6.05	8.35
4	1.40	1.70	2 40	3.10	3.66	6.22	8.58
4¼	1.44	1.75	2.48	3.19	3.77	6.39	8.81
4½	1.48	1.80	2.56	3.28	3.88	6.56	9 04
4¾	1 52	1.85	2.64	3 37	3.99	6.73	9.27
5	1.56	1.90	2.72	3.46	4.10	6.90	9.50
5½	1.64	2.00	2.84	3.64	4.32	7.24	9.96
6	1.72	2.10	3.04	3.82	4.54	7 58	10.42
6½	1 80	2.20	3.20	4.00	4.76	7.92	10.88
7	1.88	2 30	3 36	4.18	4.98	8.26	11 34
7½	1 96	2.40	3.52	4.36	5 20	8.60	11.80
8	2.04	2 50	3 68	4.54	5.42	8.94	12.26
8½	2.12	2 60	3.84	4.72	5.64	9 28	12.72
9	2.20	2 70	4 00	4.90	5.86	9 62	13 18
9½	2.28	2.80	4 16	5.08	6.08	9 96	13.64
10	2.36	2.90	4.32	5.26	6.30	10.30	14 10
11	2.52	3.10	4 64	5.62	6.74	10 98	15 02
12	2.68	3.30	4.96	5.98	7.18	11.66	15.94
13	2.84	3.50	5.28	6 34	7.62	12.34	16.86
14	3.00	3.70	5.60	6 70	8.06	13.02	17.78
15	3 16	3.90	5.92	7.06	8.50	13.70	18.70
16	3.32	4.10	6.24	7.42	8 94	14.38	19.62
17	3.48	4.30	6.56	7.78	9.38	15.06	20.54
18	3.64	4 50	6.88	8 14	9.82	15.74	21.46
19	3.80	4 70	7.20	8.50	10 26	16.42	22 38
20	3 96	4.90	7.52	8.86	10.70	17.10	23.30

SPIKES, NAILS AND TACKS.

STANDARD STEEL WIRE NAILS.						STEEL WIRE SPIKES.		
		Common.		Finishing.				
Sizes.	Length.	Diam., inches.	No. per pound.	Diam., inches.	No. per pound.	Length.	Diam., inches.	No. per pound.
2d	1"	.0524	1000	.0453	1558	3"	.1620	41
3d	1¼"	.0588	640	.0508	913	3½"	.1819	30
4d	1½"	.0720	380	.0508	761	4"	.2043	23
5d	1¾"	.0764	275	.0571	500	4½"	.2294	17
6d	2"	.0808	210	.0641	350	5"	.2576	13
7d	2¼"	.0858	160	.0641	315	5½"	.2893	11
8d	2½"	.0935	115	.0720	214	6"	.2893	10
9d	2¾"	.0963	93	.0720	195	6½"	.3249	7½
10d	3"	.1082	77	.0808	137	7"	.3249	7
12d	3¼"	.1144	60	.0808	127	8"	.3648	5
16d	3½"	.1285	48	.0907	90	9"	.3648	4½
20d	4"	.1620	31	.1019	62			
30d	4½"	.1819	22					
40d	5"	.2043	17					
50d	5½"	.2294	13					
60d	6"	.2576	11					

TABLE XXIX.

MANUFACTURERS' STANDARD LIST OF MACHINE BOLTS WITH SQUARE HEADS AND SQUARE NUTS. FINISHED POINTS.

PRICE PER HUNDRED.

ADOPTED SEPT. 20, 1899, TO TAKE EFFECT OCT. 1, 1899.

Length Inches.	¼	5/16	⅜	7/16	½	9/16 & ⅝	¾	⅞	1
1½	1.70	2.00	2.40	2.80	3.60	5.20	7.20	10.50	15.10
2	1.78	2.12	2.56	3.00	3.86	5.58	7.70	11.20	16.00
2½	1.86	2.24	2.72	3.20	4.12	5.96	8.20	11.90	16.90
3	1.94	2.36	2.88	3.40	4.38	6.34	8.70	12.60	17.80
3½	2.02	2.48	3.04	3.60	4.64	6.72	9.20	13.30	18.70
4	2.10	2.60	3.20	3.80	4.90	7.10	9.70	14.00	19.60
4½	2.18	2.72	3.36	4.00	5.16	7.48	10.20	14.70	20.50
5	2.26	2.84	3.52	4.20	5.42	7.86	10.70	15.40	21.40
5½	2.34	2.96	3.68	4.40	5.68	8.24	11.20	16.10	22.30
6	2.42	3.08	3.84	4.60	5.94	8.62	11.70	16.80	23.20
6½	2.50	3.20	4.00	4.80	6.20	9.00	12.20	17.50	24.10
7	2.58	3.32	4.16	5.00	6.46	9.38	12.70	18.20	25.00
7½	2.66	3.44	4.32	5.20	6.72	9.76	13.20	18.90	25.90
8	2.74	3.56	4.48	5.40	6.98	10.14	13.70	19.60	26.80
9	2.90	3.80	4.80	5.80	7.50	10.90	14.70	21.00	28.60
10	3.06	4.04	5.12	6.20	8.02	11.66	15.70	22.40	30.40
11	3.22	4.28	5.44	6.60	8.54	12.42	16.70	23.80	32.20
12	3.38	4.52	5.76	7.00	9.06	13.18	17.70	25.20	34.00
13	....	....	6.08	7.40	9.58	13.94	18.70	26.60	35.80
14	....	....	6.40	7.80	10.10	14.70	19.70	28.00	37.60
15	....	....	6.72	8.20	10.62	15.46	20.70	29.40	39.40
16	....	....	7.04	8.60	11.14	16.22	21.70	30.80	41.20
17	....	....	....	....	11.66	16.98	22.70	32.20	43.00
18	....	....	....	....	12.18	17.74	23.70	33.60	44.80
19	....	....	....	....	12.70	18.50	24.70	35.00	46.60
20	....	....	....	....	13.22	19.26	25.70	36.40	48.40
21	....	....	....	....	....	....	26.70	37.80	50.20
22	....	....	....	....	....	....	27.70	39.20	52.00
23	....	....	....	....	....	....	28.70	40.60	53.80
24	....	....	....	....	....	....	29.70	42.00	55.60
25	....	....	....	....	....	....	30.70	43.40	57.40
26	....	....	....	....	....	....	31.70	44.80	59.20
27	....	....	....	....	....	....	32.70	46.20	61.00
28	....	....	....	....	....	....	33.70	47.60	62.80
29	....	....	....	....	....	....	34.70	49.00	64.60
30	....	....	....	....	....	....	35.70	50.40	66.40

The following extras are to be understood as a part of the Machine Bolt List:

Bolts with Hexagon Heads or Hexagon Nuts, 10 per cent extra.

If both Hexagon Heads and Hexagon Nuts, 20 per cent extra.

Joint Bolts with Oblong Nuts, Bolts with Tee Heads, Askew *Heads, and Eccentric Heads, 10 per cent extra.*

Special Bolts with irregular Threads and unusual dimensions of Heads or Nuts will be charged extra at the discretion of the manufacturer.

AVERAGE WEIGHT OF SQUARE HEAD MACHINE BOLTS PER 100.

Length.	DIAMETER.								
	¼	5/16	⅜	7/16	½	⅝	¾	⅞	1
1½	4.0	6.8	10.6	15.0	23.0	40.5	70.0	......	......
1¾	4.4	7.3	11.3	16.1	25.1	43.7	73.1	......	......
2	4.7	7.8	12.0	17.2	26.3	44.8	76.2	......	......
2¼	5.1	8.4	12.6	18.2	27.7	47.0	79.8	......	......
2½	5.4	8.9	13.3	19.2	29.0	49.2	82.4	120.5	......
2¾	5.8	9.5	14.0	20.2	30.4	51.4	85.5	124.7	......
3	6.1	10.0	14.7	21.2	31.8	53.5	88.7	128.0	186.0
3½	6.8	11.1	16.0	23.2	34.7	57.9	95.0	137.4	196.0
4	7.5	12.2	17.4	25.2	37.5	62.3	101.3	146.8	207.0
4½	8.2	13.3	18.7	27.2	40.2	66.7	107.6	156.2	218.0
5	8.9	14.3	20.0	29.1	43.0	71.0	113.7	167.7	229.0
5½	9.6	15.4	21.4	31.2	45.7	75.4	120.0	176.1	240.0
6	10.3	16.5	22.8	33.1	48.4	79.8	126.3	184.6	251.0
6½	11.0	17.6	24.1	35.1	51.2	84.1	132.6	193.0	262.0
7	11.7	18.6	25.0	37.1	54.0	88.5	138.7	201.4	273.0
7½	12.4	19.7	27.7	39.1	56.7	92.9	145.0	209.9	284.0
8	13.1	20.8	29.5	41.0	59.4	97.2	151.2	218.3	295.0
9	......	......	33.1	45.0	64.8	106.0	163.7	240.2	317.0
10	......	......	36.7	49.0	70.3	114.7	176.2	257.1	339.0
11	......	......	40.4	53.0	75.8	123.5	188.7	273.9	360.0
12	......	......	41.0	57.0	81.3	132.2	201.0	290.0	382.0
13	......	......	......	......	86.7	140.7	213.4	307.7	404.0
14	......	......	......	......	92.2	149.2	225.9	324.6	426.0
15	......	......	......	......	97.7	157.6	238.3	341.4	448.0
16	......	......	......	......	103.1	166.1	250.8	358.3	470.0
17	......	......	......	......	108.6	174.6	263.2	375.2	492.0
18	......	......	......	......	114.1	183.1	275.6	392.0	514.0
19	......	......	......	......	119.5	191.5	288.1	408.9	536.0
20	......	......	......	......	125.0	200.0	300.5	425.8	558.0
Per inch additional	1.4	2.2	3.6	4.0	5.5	8.5	12.4	16.9	22.0

APPROXIMATE WEIGHT OF NUTS AND BOLT HEADS, IN POUNDS.

Diam. of Bolt in inches.	¼	5/16	⅜	7/16	½	⅝	¾
Weight of Hexagon Nut and Head	.017	.029	.057	.100	.128	.267	.43
Weight of Square Nut and Head	.021	.040	.069	.120	.164	.320	.55
Diam of Bolt in inches.	**⅞**	**1**	**1¼**	**1½**	**1¾**	**2**	**2½**
Weight of Hexagon Nut and Head	.73	1.10	2.11	3.73	5.6	8.75	17.0
Weight of Square Nut and Head	.88	1.31	2.56	4.49	7.0	10.5	21

PART IV.

Miscellaneous Structures.

The descriptions of the different structures given in this part have been abstracted from descriptions published in the Engineering News, Engineering Record, etc. Figs. 171 and 172 are from the Railroad Gazette; and Fig. 169 and Figs. 172 to 181, inclusive, are reproduced from originals kindly loaned by the Engineering News.

Steel Dome for the West Baden, Ind., Hotel.*

The dome of the new hotel at West Baden, Ind., is remarkable for its size. This dome has a steel framework and is larger than any other ever built, its span exceeding by about 15 feet that of the Horticultural Building of the Chicago Exposition of 1893.

The dome is about 200 feet in outside diameter and rises about 50 feet above the bed plates. Its frame consists of 24 steel ribs, all connected at the center or crown to a circular plate drum, and tied together at the bottom by a circular plate girder tie. Each rib foots at its outside end on a built up steel shoe, resting on a masonry pier. The rib is connected to the shoe by a steel pin, and the outside plate girder toe is attached to the gusset plate at this point, just above the shoe. The shoes of all the girders are constructed as expansion bearings, being provided with rollers in the usual manner.

The dome is therefore virtually an aggregation of two-hinged arches with the drum at the center forming their common connection. Their thrust at the foot goes into the circular tie-girder, and only vertical loads (and wind loads) come upon the shoes and the bearing piers. At the same time any temperature stresses are avoided, since the expansion rollers under the shoes permit a uniform outward motion of the lower ends of all the ribs.

The outline of the dome and part of the dome framing are shown

*Engineering News, Sept, 4, 1902.

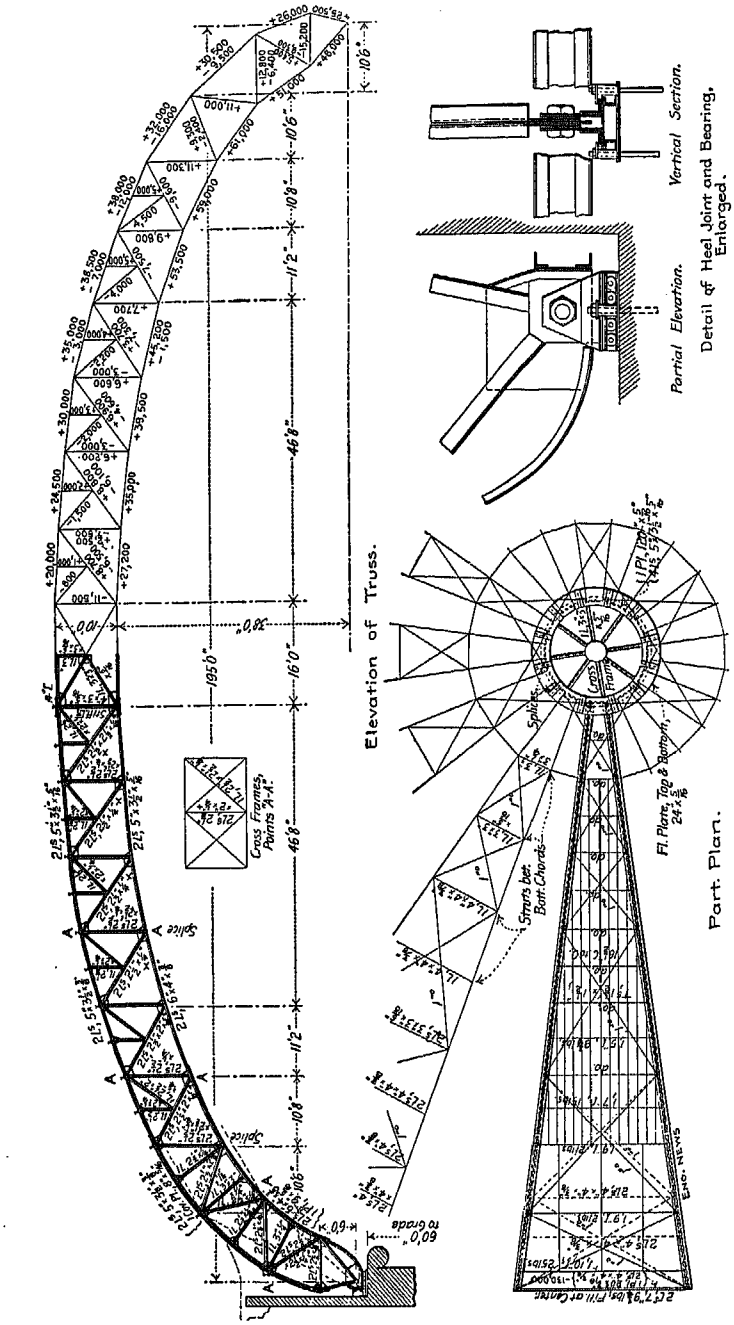


FIG. 169. STEEL DOME FOR WEST BADEN, IND., HOTEL.

in Fig. 169. The rise is between ¼ and ⅕ of the span. The outline of the top chord approximates an elliptical curve, and the bottom chord is parallel to the top chord throughout its length, except in the three end panels on either side; the depth of the arch being 10 ft. back to back of chord angles. The web members are arranged as a single system of the Pratt type, with substruts to the top chord as purlin supports. In the end sections the arrangement is necessarily modified, the sharper curvature of the chords being allowed for by more frequent strutting.

The maximum stresses in the different members of the arch are given on the right half of the rib in Fig. 169. They are obtained by properly combining the dead load stresses with the stresses due to wind blowing successively in opposite directions in the plane of the rib in question. The loads used in the calculation were a dead load separately estimated for each panel point, a variable snow load, heaviest at the center of the roof, a wind load of 30 lbs. per sq. ft. on a normal surface reduced for inclination of the roof to the vertical. The makeup of the members is given on the left half of the rib in Fig. 169. In the plan part of the dome the method of bracing the ribs is fully shown. Successive pairs of ribs are connected by bays of bracing in both upper and lower chords. In the upper chord the I beam purlins are made use of as struts, angle struts being used in the lower chord. The bracing consists throughout of crossed adjustable rods. At the center, these rods are carried over to a tangential attachment to the central drum, so as to give more rigidity against twisting at the center.

The central drum, 16 ft. in diameter by 10 ft. deep, has a web of ⅜ inch plate, with stiffener angles to which the ribs are attached. At top and bottom the drum carries a flange plate 24 ins. × 3-16 ins. for lateral stiffeners. In addition it is cross braced internally by four diametrical frames intersecting at the center. The outer tie-girder, which takes the thrust of the arch ribs, is a simple channel-shaped plate girder, 24 ins. deep, as shown on the plan. The weight of the dome complete, including framework and covering was 475,000 lbs. This makes the dead load about 15 lbs. per square foot of horizontal projection of roof surface.

Mr. Harrison Albright, of Charleston, W. Va., was the architect of the building and the design of the steel dome was worked out by Mr. Oliver J. Westcott, while in charge of the estimating department of the Illinois Steel Company. The structural steel was furnished by the Illinois Steel Co.

The St. Louis Coliseum.*

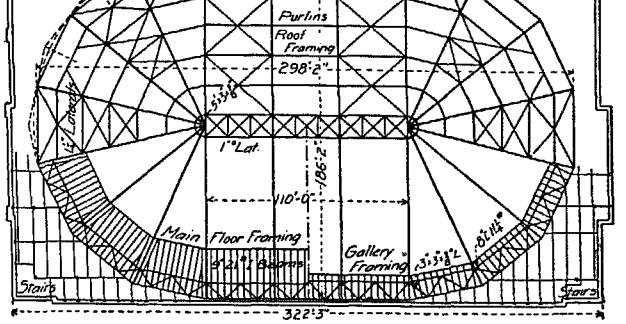
The St. Louis Coliseum Building is a rectangular brick building 186′ 2″ × 322′ 3″. The steel framework is made entirely independent of the masonry walls and consists of three-hinged arches properly braced. The Coliseum has an area of 222 × 112 feet clear of the curb wall. Ordinarily there are seats for 7,000 persons on the main floor and the galleries, but for convention purposes with seats in the arena the number can be increased to 12,000 persons.

The steel framework consists of a central arched section adjoined at each end by a half dome formed by six radial arched trusses. The main arches forming the central section have a span 178′ 6″ c. to c. of shoe pins, are spaced 36′ 8″ apart, and are connected by lateral bracing in pairs. The pins at the foot of the arches are 4 7-16″ diameter, and at the crown the pin is 2 5-16″ diameter. The rise of the arches is 80′ 0″, the lower chord points being in the curve of a true ellipse.

The end radial trusses correspond essentially to the semi-trusses of the main arches except for their top connection, where their top chords are attached to a semicircular frame supported by the end main trusses and designed to receive thrust, but no vertical reaction, as shown in Fig. 170.

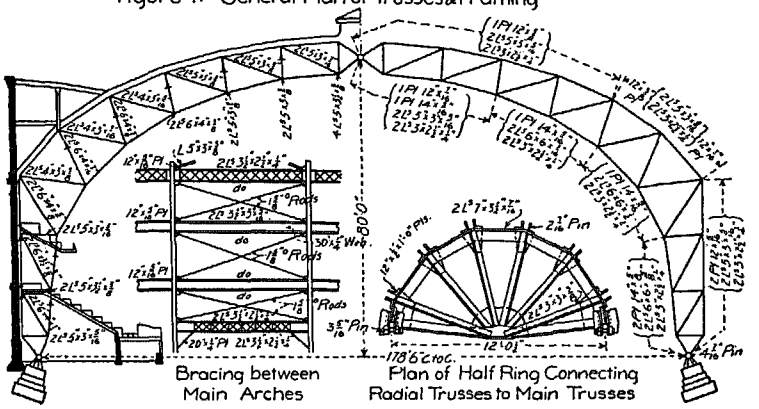
The roof covering of asphalt composition is laid on 1⅜-inch boards, resting on 2½ × 16-inch wood joists, 3 feet apart and ceiled underneath. These, in turn, are carried by the steel purlins of the structure, which are spaced about 16 feet apart. The gallery floor beams are carried on stringers of 8-inch channels spaced 3′ 8″ center to center, carried by girders running between, and supported by the arches. The rear stringer is a plate girder; the front one is a latticed girder, the gallery beams running through the latter and cantilevering out 5′ 4″. The main floor beams, supporting the lower tier of seats, consist of 9-inch I beams, spaced 3′ 8″ center to center, which are similarly carried on girders, and their lower ends rest on a brick wall.

*Engineering News, Aug. 10, 1899, and Engineering Record, 1899.



Sectional Plan

Figure 1. General Plan of Trusses & Framing



Bracing between Main Arches

Plan of Half Ring Connecting Radial Trusses to Main Trusses

Main Arches

Figure 2 - Cross Section Showing Construction of Main Arches

FIG. 170.

The loads, in accordance with which the trusses were figured, are as follows:

CASE I.

Wooden deck and gravel of roof	17.5	lbs. per sq. ft., vertically
Steel	12.5	" " " " "
Snow and wind	25.0	" " " " "
Total	55.0	" " " " "

Add for floors, viz.:

Main floors, banks and galleries	105.0	" " " " "
Attic floors	60.0	" " " " "

CASE II.

Wooden deck and gravel of roof	17.5	lbs. per sq. ft., vertically
Steel	12.5	" " " " "
Snow	10.0	" " " " "
Total	40.0	" " " " "

Wind pressure over entire elevation of wall and roof of 30.0 lbs. per sq. ft., horizontally

LOADS ON PURLINS.

Wooden deck and gravel of roof	17.5	lbs. per sq. ft., vertically
Steel	3.5	" " " " "
Snow and wind	25.0	" " " " "
Total	46.0	" " " " "

LOADS ON FLOOR BEAMS, GIRDERS AND COLUMNS OF MAIN FLOORS.

Banks and galleries, beams	140	lbs. per sq. ft.
Banks and galleries, girders	112	" " " "
Banks and galleries, columns	105	" " " "
Attic floors beams, columns and girders	60	" " " "

For the main trusses, in addition to the stresses of Case II., there was added the stress due to the wind bracing between these trusses.

For the radial trusses, in addition to loading of Case II., there was assumed an additional load of 50,000 pounds supposed to act up or down at the upper point of truss; this load being what was assumed probable in case there was slight unequal settlement of the footings.

For the half ring connecting the tops of the radial trusses there was another case assumed, beside Cases I. and II.—viz., a thrust of 50,000 pounds at any point of the half ring; this being the thrust of a radial truss under its full live and wind load.

All the material used was of medium steel, excepting the rivets, which were made of soft steel. Both material and workmanship conform to manufacturer's standard specifications.

UNIT STRAINS.

Tension	16,000 lbs. per sq. in.
Compression, for lengths of 90 radii or under	12,000 " " " "
Compression, for lengths of over 90 radii	17,100—57 $l \div r$
Combined stress due to tension or compression and transverse loading	16,000 " " " "
Shear on web plates	7,500 " " " "
Shear on pins	11,000 " " " "
Shear on rivets	10,000 " " " "
Bearing on pins	22,000 " " " "
Bearing on rivets	20,000 " " " "
Bending, extreme fibre of pins	25,000 " " " "
Bending, extreme fibre of beams	16,000 " " " "

Lateral connections have 25 per cent greater unit strains than the above.

In Case II of trusses, the above unit strains were increased one-third. The main and radial arch trusses are built as shown in Fig. 170, except that above the haunches the ribs of the radial arches are T-shaped instead of I-shaped, i. e., they have no inside flanges. The purlins are triangular trusses 4½ feet deep, made of angles. The bracing between main arch trusses terminates at the bottom with heavy portal struts of triangular box section. The lateral rods are not carried to the ground on account of the obstruction they would make. The radial trusses are coupled together in pairs with lateral rods down to the ceiling line. The thrust due to wind is transmitted from them into the line of girders around the structure at this point, and into the adjoining floor systems. The compression ribs of the main and radial arches are stayed laterally by angle iron ties, connecting to the first panel-point in the bottom chord of the purlins. In the planes of the first diagonal braces of the trusses above the haunches, diagonal rods connect the bottom ribs of the trusses to the upper ribs of the next trusses. No struts were used between the bottom chords, as they would have been directly in the line of vision from the rear gallery seats to the farther end of the arena. The front and rear girders supporting the gallery and main floor beams are tied together with a triangular system of angle iron bracing.

To provide for expansion, the radial purlins and all the girders between the arches have slotted hole connections in every alternate bay. The diagonal rods between the two lines of ridge purlins were tightly

adjusted on a hot day. To prevent secondary strains in the half ring to which the radial trusses are connected at their tops, there is 1-16-inch clearance in all the pin holes. There is also clearance between the pin plates, so that the trusses and the ring can slide a little sideways on their pins. The lines of the arch trusses were laid out full size and the principal points checked by independent measurements in the template shop, and the work was accurately assembled. In order to avoid the handling of large, heavy pieces before the drill press, the foot of the arch, through which the pin hole was bored, was made separately and afterward riveted on.

The total weight of the iron in the entire structure was 1,905,000 pounds, as follows: Main arches, 64,000 pounds, each; radial arches 21,000 pounds, each; purlins between main trusses 1,450 pounds, each; main floor stringers 810 pounds, each; balcony floor stringers, 280 pounds, each; cast shoes 3,000 pounds, each. There were 4,188 days labor spent on the work in the shop and 3,550 days labor during erection, the average number of men in the erecting force being about 50. The stress diagrams and detail plans of the steel frame were made under the supervision of Mr. Stern, in the office of the Koken Iron Works, who were contractors for the ironwork, and were submitted for approval to the consulting engineer, Mr. Julius Baier, Assoc. M. Am. Soc. C. E. Mr. C. K. Ramsey was the architect of the Coliseum, and Mr. L. H. Sullivan was the consulting architect. Mr. A. H. Zeller was consulting engineer for the Board of Public Improvements; Mr. J. D. McKee, C. E., was shop inspector, and the Hill-O'Meara Construction Company was the general contractor.

The Locomotive Shops of the Atchison, Topeka and Santa Fe R. R., Topeka, Kas.*

This building is intended for all the locomotive work, including boilers and tenders. It is of particular note for its great size and the peculiar features of its design. In general plan it is 852 ft. long and 153 ft. 10 ins. wide, the width being divided into a center span of 74 ft. 3 ins. and two side spans of 39 ft. 9 ins. It is of self-supporting steel frame construction, with concrete foundations and floor, 13-in. brick walls, and Ludowici tile roof. There is no sheathing under the tiles, which thus constitute the sole covering. The tiles are laid on 2 × 2-in. timber strips to which every fourth tile is fastened by copper wire.

The most striking feature of the design is that the saw tooth or weaving shed type of roof is adopted for the side spans, the glazed vertical sides of the ridges facing northward. This feature was introduced with the view of making the shop as light as possible. The arrangement could not well be used where heavy snows are frequently experienced, as the snow would pack between the ridges, but there are comparatively few heavy snow storms in the vicinity of Topeka. In addition to this arrangement, the greater proportion of the area of the side walls is composed of windows, while the exposed parts of the sides of the central span (between the ridges of the side spans) are also glazed. There are also several windows in the end walls. The roof of the central span has on each side of the ridge a skylight 12 ft. wide, extending the full length of the building. These skylights are fitted with translucent fabric instead of glass. By these various means an exceptionally good lighting effect and diffusion of light are obtained and the shop is in fact remarkably light even on a gloomy day. There is no monitor roof, but ventilation is provided for by Star ventilators 25 ft. apart along the ridge of the main roof.

The columns are built up of pairs of 15-in. channels, and independent columns of similar construction carry the double-web box girder runways for the electric traveling cranes which run the entire length of the central span. Fig. 173 shows the elevations, sections and plans

*Engineering News, Jan. 3, 1903; and Railway Gazette, Nov. 7, 1902.

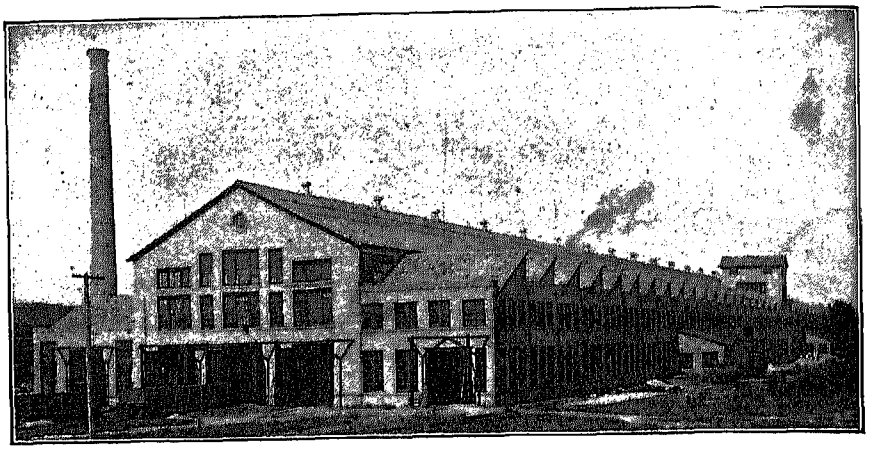


FIG. 171. LOCOMOTIVE SHOP.

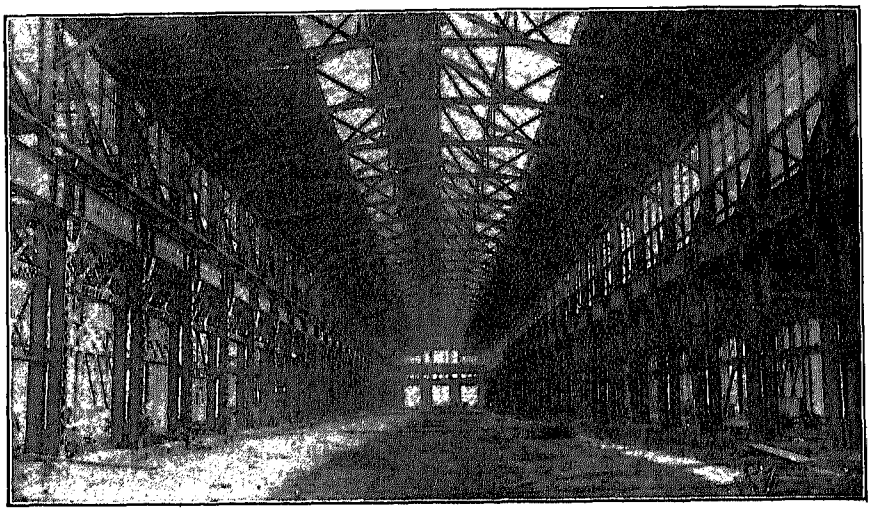


FIG. 172. CROSS-SECTION LOCOMOTIVE SHOP.

of the steel structural framework, and Fig. 174 is a partial elevation on the east side. Fig. 175 shows the design of the central roof trusses and the lattice girders which form longitudinal bracing between the trusses. This longitudinal bracing is not continuous but is fitted only between

alternate pairs of trusses. End trusses are built into the walls, as these walls are pierced by numerous windows and doors and are not relied upon in any way to support the roof. Portal bracing is fitted between the side or wall columns at intervals. No metal less than 3/8-in. thick is used in the structural work.

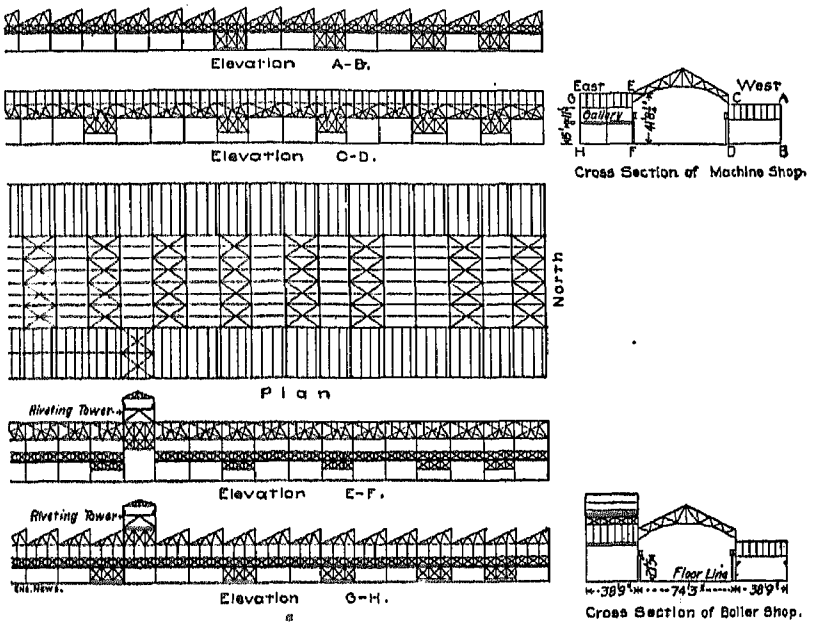


FIG. 173. PART ELEVATIONS AND PLANS OF STEEL STRUCTURAL WORK OF NEW LOCOMOTIVE SHOPS.

The roof trusses are proportioned for a load of 15 lbs. per sq. ft. for the weight of the roofing, 10 lbs. per sq. ft. for snow, and 25 lbs. per sq. ft. for wind pressure, or 50 lbs. per sq. ft. in all. The members

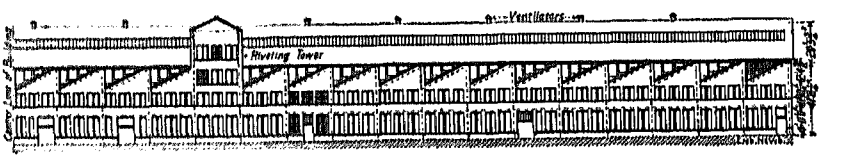


FIG. 174. HALF EAST ELEVATION OF NEW LOCOMOTIVE SHOP. (SHOWING RIVETING TOWER AND WEAVING SHED ROOF.)

shop was built at the Toledo Works of the American Bridge Co. The steel is painted a light grey, and the brick is whitewashed, a pneumatic machine being used for the latter work.

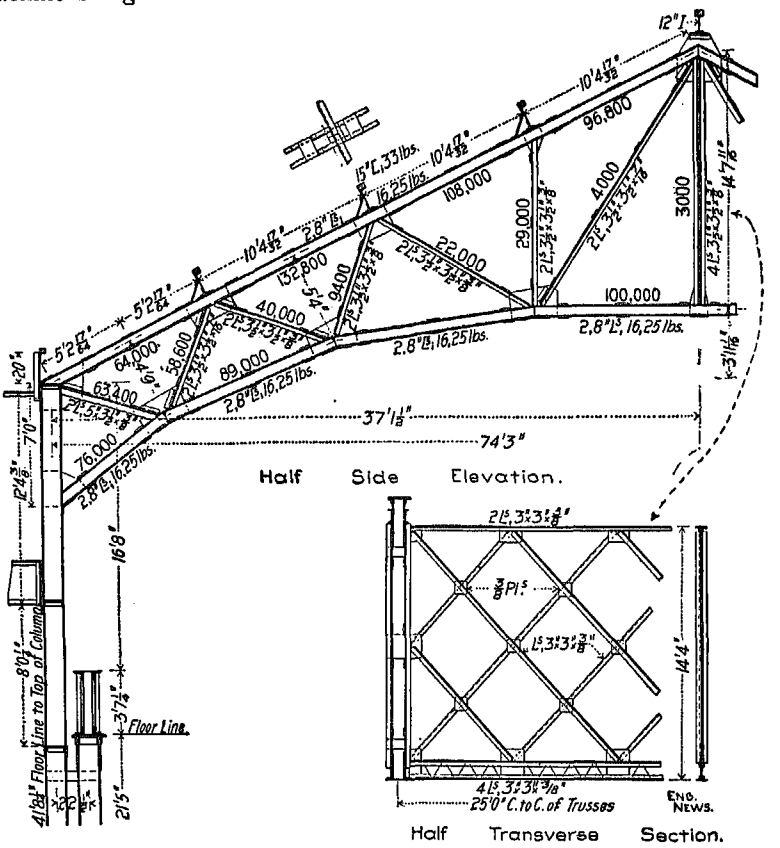


FIG. 175.

The arrangements for lighting the shop by day have already been referred to. For night work there will be arc lights for general lighting and incandescent lamps convenient to the tools, etc. The building is heated by the Sturtevant hot blast system. On each side are two fan rooms, each containing a steam-driven blower fan and a heating

chamber filled with coils of pipe through which passes the exhaust steam. The hot air is delivered into two longitudinal underground conduits parallel with the lines of columns, with a duct leading to the surface at each column. Each duct is fitted with a vertical sheet iron pipe 7 ft. high, with a flaring head to deliver the air horizontally. The plant is guaranteed to maintain a temperature of 70° F. throughout the shop in zero weather.

The floor foundation is formed of 6 inches of concrete resting on the natural soil well tamped. The concrete is composed of 1 part Louisville cement, 2 parts sand and 4 parts stone. On the concrete are laid yellow pine nailing strips, 3" × 4", 18 ins. c. to c., to which is spiked the 1¾-in. splined hard-maple flooring. All tracks in the shop are laid with 75-lb. rails on ties of New Mexico pine treated by the zinc-chloride process, the floor concrete being laid only to the ends of the ties, so that adjustment of the track can be made without tearing up the floor. At the engine pits (which are of concrete) the rails are laid on longitudinal timbers. The concrete for column foundations is composed of 1 part Iola Portland cement, 3 parts sand and 5 parts stone. These foundations are 8 to 15 ft. deep, extending to solid clay. They are built up with gas pipe sleeves to form holes for the anchor bolts, and the holes in the bed plates of the columns are slotted longitudinally so as to allow of adjustment for any slight variation. The foundations for the tools, etc., are also of Portland cement concrete, and these are built by the mechanical department to suit its own requirements as to arrangement of tools. This arrangement was only arrived at after careful study, and of course no changes can be made without expensive work in cutting out and replacing concrete. One suggestion for the floor construction was to use a brick floor with no concrete, so as to allow for future changes and putting in new foundations.

The Locomotive Erecting and Machine Shop, Philadelphia & Reading R. R., Reading, Pa.*

The combined machine and erecting shop is 204 ft. 4½ ins. wide and 749 ft. 10 ins. long, with provision made for its extension to a total length of 1,000 ft. At its present length it has repair pits for 70 locomotives, and the proposed extension will provide for 30 pits more. Fig. 176 shows the general arrangement of the shop in plan, and Fig. 177 is a transverse section showing the general character of the construction.

The building, it will be observed, is divided transversely into three bays by means of two rows of intermediate columns running lengthwise of the building. These intermediate columns and the side wall columns carry the roof trusses and the overhead cranes, and are spaced 20 ft. apart longitudinally. The walls of the building are entirely independent of the steelwork. Considering the building transversely it will be observed that the two side bays contain the repair pits; one pit be-

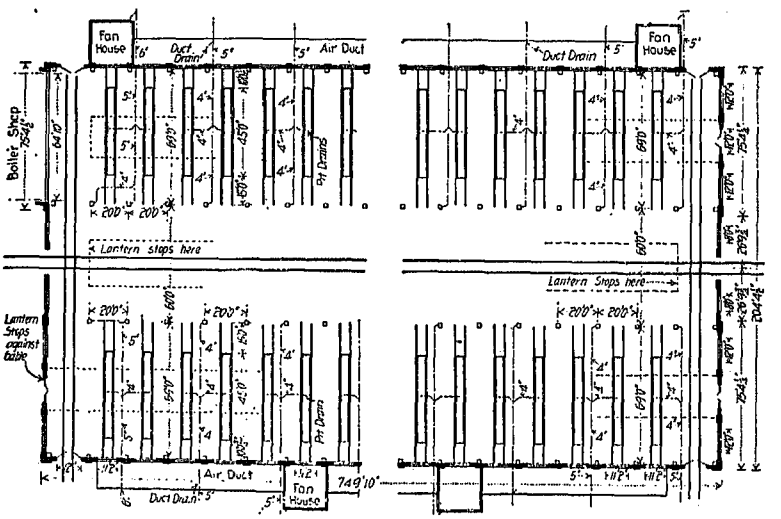


Fig. 176. Part plan, showing pit and column arrangement.

*Engineering News, May 24, 1900.

tween each pair of columns, 35 pits on each side spaced 20 feet apart c. to c. Over each side bay there are two traveling crane tracks, one above the other. The top track will be for a crane of 120 tons capacity, capable of lifting a locomotive and carrying it the entire length of the building at a height great enough to clear the locomotives standing on the repair pits beneath. On the lower track there will be placed a number of 35-ton cranes for handling parts of locomotives and material. The middle bay composes the machine shop, and it is also covered by a track for a number of traveling bridge cranes of 10 tons capacity each. A material track runs lengthwise of the middle bay at the center, and for bringing the locomotives into the shop there is a track extending transversely through the shop at each end. These transverse tracks connect by means of turntables with a yard track running parallel with the east side of the shop.

Taking up the construction in detail, the steelwork is the first part perhaps to demand attention. As already stated, the nature and general arrangement of the steel frame is shown by Figs. 176 and 177. It may be divided into two parts for description. The first part comprises

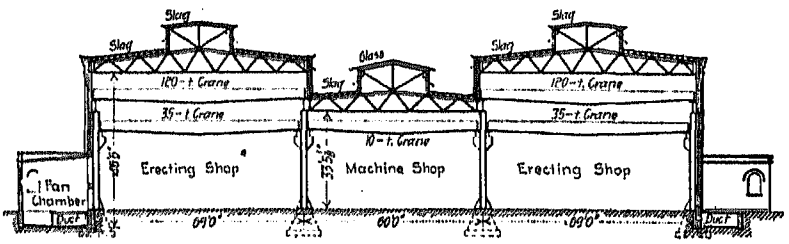


FIG. 177. TRANSVERSE SECTION OF FRAMEWORK.

the main columns and the traveling crane track system which they support directly, and the second part comprises the roof framing. Considering the construction of the main columns first, it will be seen that it had to be devised to withstand an unusually severe combination of loads, including not only the dead and wind loads usual to all building frames, but also a very heavy concentrated moving load from the traveling cranes. Careful attention was also required to provide means for longitudinal expansion of the columns and crane track girder system and still preserve the utmost stiffness possible in the heavily loaded crane tracks.

The interior and wall columns were substantially similar in construction, both being composed of two channel sections made up of plates and angles, held together by a double lacing of 2½ × ½-in. bars. Fig. 180 shows the column construction in detail. Each column is founded on a concrete pier with a granite capstone. The concrete in these piers and for all other foundation work was composed of 1 part Portland cement, 2 parts clean sharp sand and 5 parts stone broken to pass a 2-in. ring. In making the concrete the sand and cement will first be mixed dry and then wet, and the wet mortar will then be mixed with the broken stone as it comes moist from washing. The shape of the column piers is indicated in Fig. 177, and the columns are fastened to them by four anchor bolts for each column. The columns are finished by milling machine at both top and bottom, and are capped and stiffened at the tops to provide a bearing for the 4-ft. 1⅜-in. plate girders carrying the 120-ton crane tracks.

The track construction for the bridge cranes is shown in detail in Figs. 177 and 180. As already stated, the tracks for the 120-ton cranes are carried on plate girders resting directly on the tops of the columns, and running lengthwise of the building. These track girders are 4 ft. 1⅜ ins. deep back to back of flange angles and have spans of 20 ft., c. to c. of columns. They are milled square at the ends and the rivets of the end angles have flat heads. To allow for expansion the ends are not butted close together, but are separated by a clear space of ¼-in., by the construction shown in Fig. 180. From the drawing it will be seen that each girder span has one fixed and one expansion end, the expansion being provided for by the slotted rivet holes in the bearing plate and by the space between the ends of the girders. An exactly similar expansion end construction is provided for the girders carrying the tracks for the 35-ton and 10-ton cranes. These three tracks are on the same level and the girders supporting them are carried by brackets on the main columns. This bracket and girder construction is shown in Fig. 177.

The crane tracks proper consist of ordinary railway rails laid directly on the special cover plate forming part of the top flange of each track girder, to which they are attached by stamped steel clips riveted to the cover plate. For the 120-ton cranes the rails weigh 150 lbs. per yard, for the 35-ton cranes they weigh 85 lbs. per yard, and for the 10-ton cranes they weigh 70 lbs. per yard. The joints are located over the expansion joints at each column and are spaced ⅛-in. open, the angle

splices for the rails being arranged to permit expansion and contraction. The rails were rolled to the recommended Am. So. C. E. sections for their respective weights. The specifications require that the crane track rails must be in perfect alinement horizontally and vertically and that the gage must not vary more than ¼-in. at the maximum. To obtain vertical alinement it is specified that not more than ⅛-in. thickness of shims, each the full size of the bearings, shall be placed between the column top and the girder. The idea, it will be seen, has been to provide for the heavy rolling loads by substantial construction and accurate workmanship, and to keep the track smooth and rigid by distributing the expansion over a number of joints instead of having all the allowance made for it at one or two points.

Turning now to the roof framing, it will be seen from Fig. 177 that there is a separate roof over each longitudinal bay of the building. The roof trusses for the middle bay rest directly on the tops of the intermediate rows of columns, but those for the two side bays are carried by special roof columns rising from the main columns, as shown by Fig. 180. The trusses in each bay are connected by purlins and lateral bracing and carry lantern roofs with glazing. Details of the roof trusses, bracing and lantern roof construction are shown in Fig. 178 so fully as to make any further description unnecessary. The roof covering and the glazing in the lanterns, however, deserve brief special notice.

The roof covering consists first of a 1 × 8-in. hemlock sheathing, having its upper surface planed. This sheathing is to be covered by four thicknesses of roofing felt spread with granulated slag. The construction is specified to be as follows: Make the outside course next the edge of five thicknesses of felt, then lay each succeeding layer at least three-fourths of its width over the preceding layer, firmly securing it in place, and thoroughly mop the surface underneath each succeeding layer as far back as the edge of the next lap with a thin coating of roofing cement. This cement is in no case to be applied hot enough to injure the wooly fibre of the felt. At least 70 lbs. of felt must be used per 100 sq. ft. of roof. Over the entire surface of the felt laid as described there is to be spread a good coating of cement, not less than 10 gallons (including what is used between the layers of felt) of cement being employed per 100 sq. ft. of roof. This cement coating is to be covered with a coating of slag, granulated and bolted for the purpose, using no slag larger than will pass through a ⅝-in. mesh and none smaller than will be caught by a ¼-in. mesh. This slag must be free

from sand, dust and dirt and must be applied perfectly dry while the cement is hot.

The structural features of the glazing in the windows and lights of the monitor roofs are fully shown by the drawing of Figs. 177 and 178. It will be noted that the sash in the side walls of the monitor roofs are hung on pivots at the middle of each side. There is a similar arrangement of sash in the upper side wall windows of the side roofs overlooking the low middle roof, except that the sash in alternate bays only are pivoted. These latter sash are to be provided with "Brand's" sash openers, with chains operated from the crane runways and arranged to open all the sash of one bay at one time. The feature most worthy of note, however, is that the sash in the side walls of the monitor roofs are to be operated by compressed air power from a central point on one of the side walls of the building near the floor. The sash will be operated in sections about 75 feet long and the piping and valves will be so arranged that any one or all of the sections can be operated at one time.

Next to the framework of the building, the structural features of most interest are perhaps the pit and floor construction. This is shown in detail by Fig. 180. In constructing the floor the ground inside the building will be carefully leveled and well tamped and puddled if necessary. The entire area of the main building, except under the pits, where cement concrete will be used, and under the railway tracks, where stone ballast will be used, will then be covered with a layer of bituminous concrete. This concrete will be composed of well screened cinders and No. 4 coke oven composition mixed in the proportion of at least one gallon of composition for each cubic foot of cinders. This composition is to be laid hot and well rammed. Yellow pine floor sleepers, 6 × 6 ins. square, will be bedded in the bituminous concrete at intervals varying from 4 ft. to 5 ft. transversely of the building. To these stringers there will be spiked an under floor, or lining, of hemlock plank planed on both edges and on the upper side. On top of this lining there will be laid a flooring of 1⅛ × 4-in. maple boards of a uniform and regular width, planed on top and both edges, worked on the back to a depth of 1-16-in. and a width of 2¾ ins., laid across hemlock floor, bored on a slant for nailing and face nailed with 12d. nails, the two lines of nails in each board being staggered. The nails will be placed in lines on each side of the board and not over 16 ins. apart, and the nails in one line will be opposite the middle point between the nails in the opposite line.

tions for the main columns, Fig. 180, and have a facing of 1 in. of cement. It will be noticed that the bottoms of the pits are crowned transversely and have longitudinal side drains of "catch basin" type, running into gutters which connect with the tile drain pipe system of the building. The pit rails are carried on 10 × 12-in. yellow pine side timbers resting on the concrete pit walls and connected by anchor rods extending into the bituminous concrete floor foundation. The railway tracks in the building consist of 80-lb. rails spiked to cross-ties laid in stone ballast.

Before taking up the exterior construction of the building, a brief reference may be made to the method adopted for heating it. By referring to Figs. 176 and 177 it will be noticed that an underground duct extends along each side of the buildings. This duct will contain all the steam, air and other piping entering the building and will also serve as a hot-air duct for heating the building. For the latter purpose the duct connects with fan houses placed at intervals along the side of the building, there being four fan houses on the west side and three on the east side. It is expected that the exhaust steam will be ample to heat the building. The general principle of the operation of the heating apparatus is to use the air over and over again, depending upon the natural ventilation to keep it fresh. Structually the ducts are simple, their bottoms and side walls being of concrete with a cement plaster, and the roof being expanded metal and concrete supported by transverse roof beams. The roofs of the fan houses will consist of I beams carrying T-iron purlins holding book tile, which in turn carry a felt and slag roofing similar to that already described for the main building.

From the structural point of view, the wall construction of the main building presents nothing that is particularly notable. These walls are of brick masonry resting on a concrete footing and are anchored to the wall columns, as shown by Fig. 178, but they are independent structures, receiving no support from these columns. The entire wall area is pretty well taken up with doors and windows, there being one in each panel or between each pair of wall columns. This introduces a considerable amount of arch work, but nothing that is of unusual character.

To illustrate the simple methods which have been adopted to secure a pleasing exterior appearance for the building, the typical details of the wall construction are shown in Fig. 179.

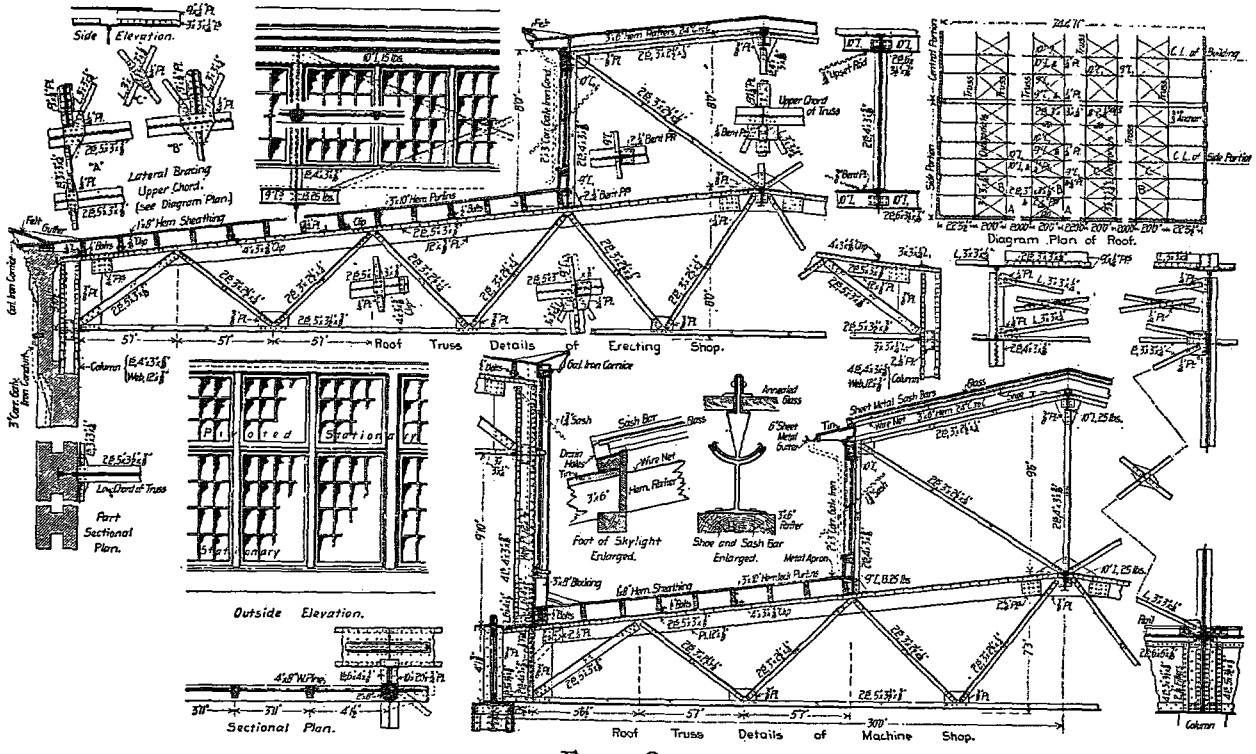

Side Elevation.
Lateral Bracing Upper Chord. (see Diagram Plan.)
Roof Truss Details of Erecting Shop.
Diagram Plan of Roof.
Part Sectional Plan.
Outside Elevation.
Sectional Plan.
Foot of Sky-light Enlarged.
Shoe and Sash Bar Enlarged.
Roof Truss Details of Machine Shop.

FIG. 178.

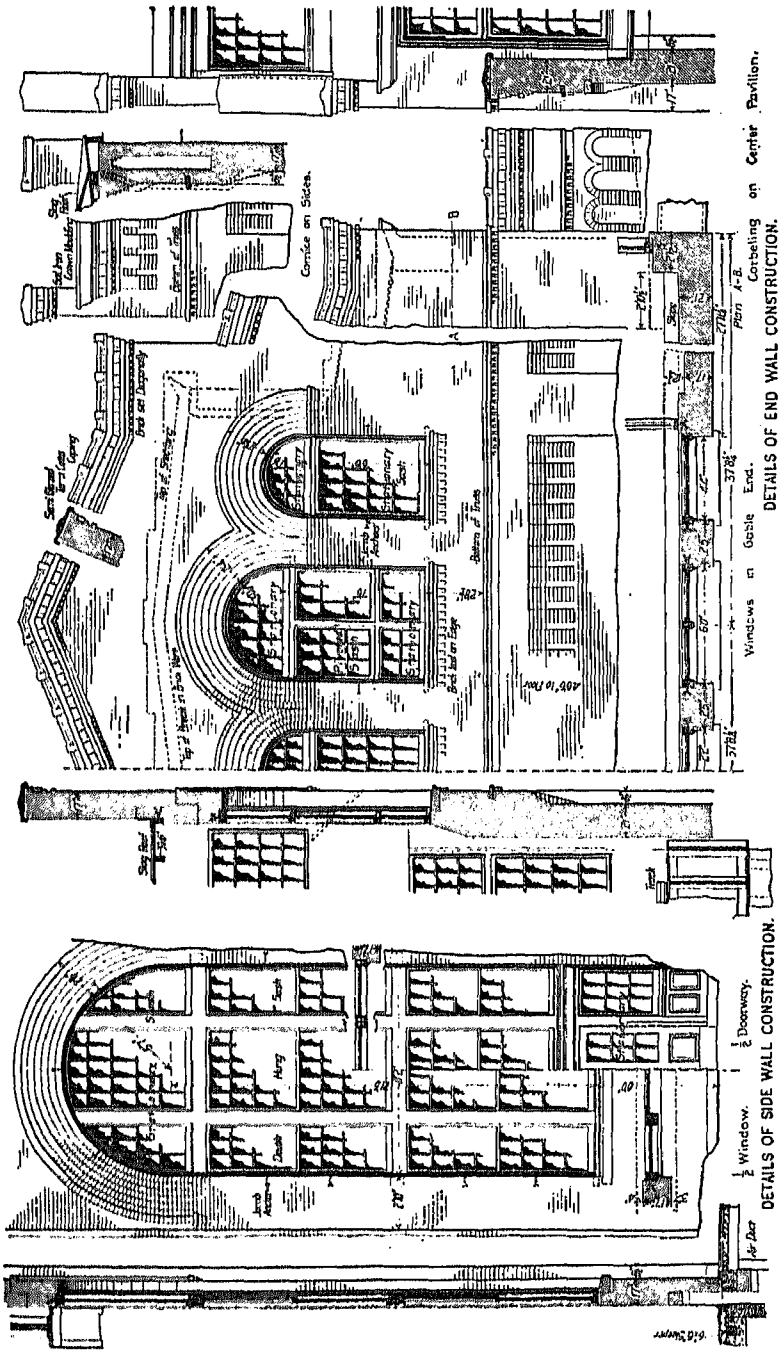

DETAILS OF END WALL CONSTRUCTION.
Windows in Gable End.
Corbeling on Center Pavilion.
DETAILS OF SIDE WALL CONSTRUCTION.
½ Window.
½ Doorway.

FIG. 179.

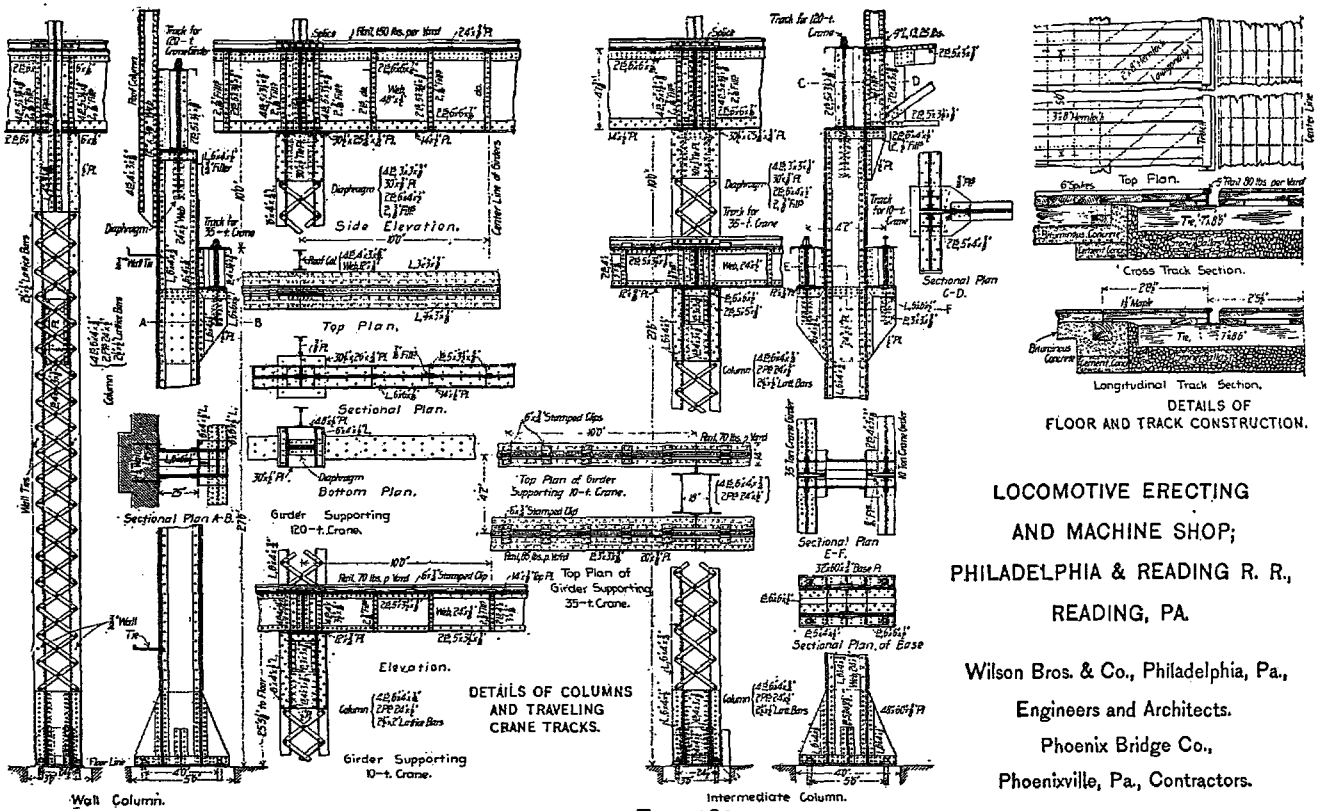


FIG. 180.

LOCOMOTIVE ERECTING AND MACHINE SHOP; PHILADELPHIA & READING R. R., READING, PA.

Wilson Bros. & Co., Philadelphia, Pa., Engineers and Architects.

Phoenix Bridge Co., Phoenixville, Pa., Contractors.

The New Steam Engineering Buildings for the Brooklyn Navy Yard.*

The new buildings constructed for the steam engineering department of the Brooklyn Navy Yard, at New York City, to take the place of the buildings destroyed by fire on Feb. 15, 1899, consist of a machine shop, an erecting shop and a boiler shop, arranged to occupy a U-shaped area. The erecting shop is 130 × 252 ft., the machine shop is 130 × 350 ft., and the boiler shop is 96 × 300 ft.

The general framework details of the machine, and the erecting shops are shown by Fig. 181. The construction of the boiler shop is similar, but the dimensions are, of course, smaller. Referring to the drawings of Fig. 181, it will be observed that the buildings are divided transversely into three bays, a center bay 70 ft. wide and two 30-ft. side bays. The two side bays are covered by shed roofs, above which rise a clerestory and gable roof to cover the center bay. The side wall columns are 12-in. I beams filled between with a brick wall for a height of about 4 ft., and above this point covered with glazing to the cornice line. The intermediate columns are of lattice and channel construction, and are 43 ft. 6 ins. high, reaching to the level of the junction of the shed roof and the clerestory. They carry between them, longitudinally of the buildings a double intersection truss 15 ft. deep, and support a box girder crane runway and the roof columns of the clerestory.

The framework is entirely of steel of from 60,000 lbs. to 70,000 lbs. ultimate strength, and an elastic limit of one-half the ultimate strength. The specifications for workmanship and other essentials correspond to ordinary first-class practice in these respects. The notable feature of the building is the extent to which glazing has been employed in the side walls and roofing, rather than in any novelty in the framework structure.

The foundations consist of concrete column pedestals carried on piles, the column base plates being anchor-bolted directly to the concrete. The concrete used was mixed according to the Navy Yard specifications, which require that the cement exceed the voids in the sand about 25%, and that the dry mixture of cement and sand exceeds

*Engineering News, April 25, 1901.

the voids in the aggregate about 25%. The piles were driven to a final penetration of 1 in. under a 3,000-lb. hammer falling 15 ft. The usual requirements as to soundness of timber and character of cement, sand, aggregate and water were enforced.

The side wall construction for the shed bays, as already stated, consists of a brick base wall with glazing above. This glazing is put

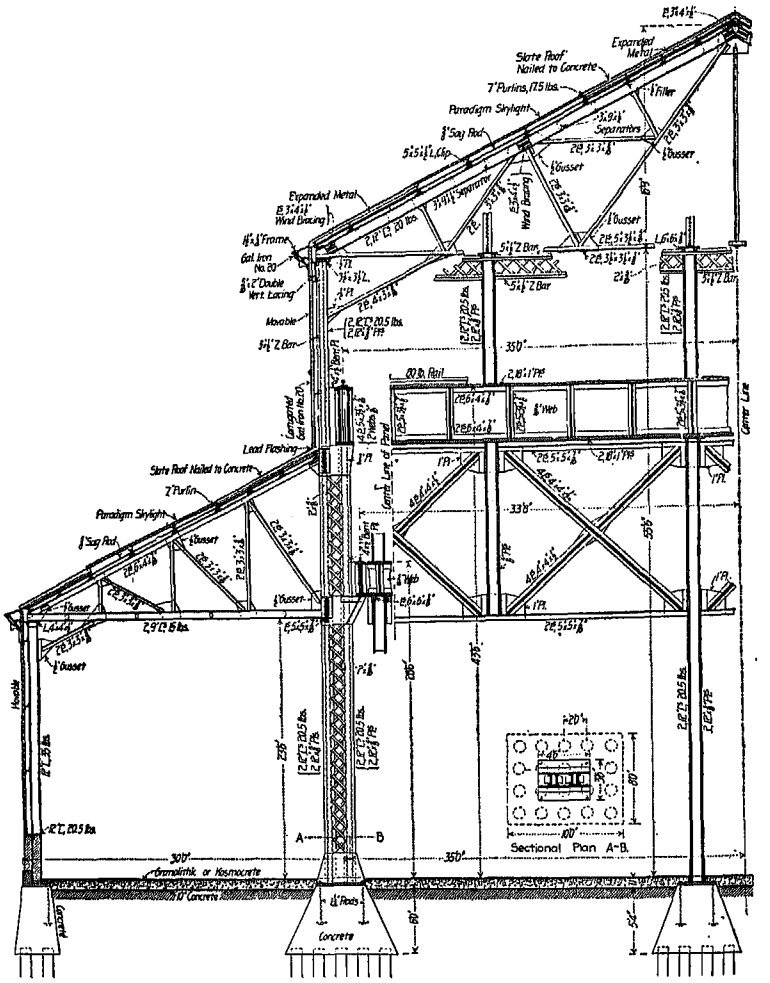


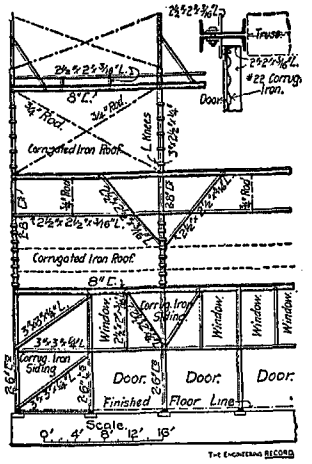
FIG. 181. HALF TRANSVERSE SECTION SHOWING STEEL FRAME DETAILS, STEAM ENGINEERING BUILDINGS, BROOKLYN NAVY YARD.

on in panels or sections; each is capable of being swung out to provide ventilation. The side walls of the clerestory consist of corrugated iron covering for the lower part, and glazing above. The bulk of the area of the shed roofs is skylight, and wide skylights are also placed in the clerestory roof. Fully 60 per cent of the area of the external walls and roof is glazed.

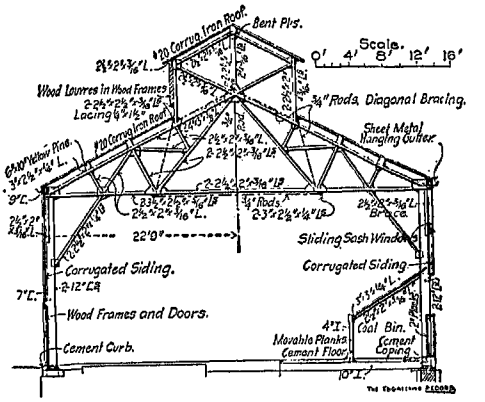
The large proportion of glazing to the total wall and roof area makes the interior of the shops extremely well-lighted. The entire glazing is on the Paradigm skylight and side light system and was carried out by Arthur E. Rendle, of New York City, who controls the system.

The entire construction is of incombustible materials. There is no wood used in the building, except for the body of the doors, and here it is covered with tin plate. In this respect, and in respect to the amount of glazing used, the buildings are somewhat remarkable, even among modern shop buildings.

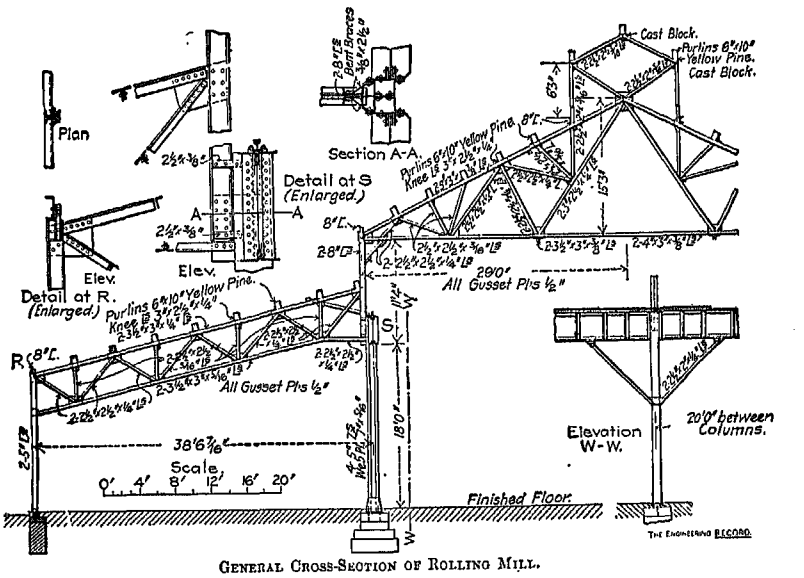
Turning now to some of the special details, it will be noticed from Fig. 181 that a somewhat unusual construction has been adopted for the main columns. Each column consists of two main members, each composed of two channels riveted back to back, the two main members being connected by double latticing. The resulting section is rather remarkable for its length, as compared with its width, but its purpose was evidently to give ample flexural strength to withstand eccentric load of the bridge crane girders. Details of the crane girders and crane tracks are shown by Fig. 181; there being one 40-ton crane and two 10-ton cranes. It will also be observed from Fig. 181 that the lower portion of the clerestory roof has a concrete and expanded metal covering with roofing slate nailed direct to the concrete. The concrete is composed of Portland cement and cinder, and is 3½ ins. thick. The floor construction throughout will be 10 ins. of concrete covered with 1 in. of granolithic or Kosmocrete. In the tool and testing rooms off the machine shop, and between the boiler and erecting shops, an effort has been made to secure a dust-proof construction, and a roof which will be free from drippings due to condensed moisture. Over these rooms the skylight roofs consist of double-glazed Paradigm skylights, with a 1-in. air space betwen them.



PART SIDE ELEVATION.



CROSS-SECTION OF BOILER HOUSE.



GENERAL CROSS-SECTION OF ROLLING MILL.

FIG. 182.* STEEL ROLLING MILL BUILDING FOR THE AMERICAN ROLLING MILL COMPANY, MIDDLETOWN, OHIO.

*Engineering Record, July 20, 1901.

Government Building, St. Louis Exposition.*

The Government Building at the St. Louis Exposition had a steel framework with steel arch trusses of the three-hinged type. The span of the arches is 172 ft. c. to c. of pins, and their rise from heel pins to center pins is 66 ft. 9½ in. The trusses are spaced 35 ft. apart, and are connected laterally by six lines of lattice girders, carrying the posts of the main roof and monitor roof, and by eight other intermediate transverse struts. A horizontal wind strut truss is located as shown in Fig. 184.

The assumed loading was as follows: Dead Load: Roof, 10 lbs. per sq. ft. on slope; dome, 12 lbs. per sq. ft. of roof surface; side walls of dome and building, 15 lbs. per sq. ft. of surface; trusses, 10 lbs. per sq. ft. of floor surface.

Wind Load: Side walls, 20 lbs. per sq. ft.; dome, curved, 15 lbs. per sq. ft. of projection on a vertical surface.

Snow Load: On roof of main building, 20 lbs. per sq. ft. horizontal; reduced to 10 lbs. per sq. ft. on ventilator over center.

The revised estimate of loads for a 35-ft. bay figured out as follows, per sq. ft. of horizontal projection:

	Lbs. per sq. ft.
Weight of steel	13.1
" " roof	6.6
" " tin covering	0.5
Actual roof	20.2
Calculated total	21.5

The loading on one truss for the 35-ft. bay was:

	Actual Weight in lbs.	Estimated Weight in lbs
Total Dead Load	40,500	70,000
Total Steel	80,000	64,000
Grand Total	120,500	134,000

The arches are built up of channels, plates and angles, and have 4½-in. shoe pins and 3-in. center pins. The shoe pins of each truss are connected by a tie bar consisting of a line of 9-in. I-beams. The stresses in the arches are given in Fig. 183, while the details are shown in Fig. 184.

* Engineering News.

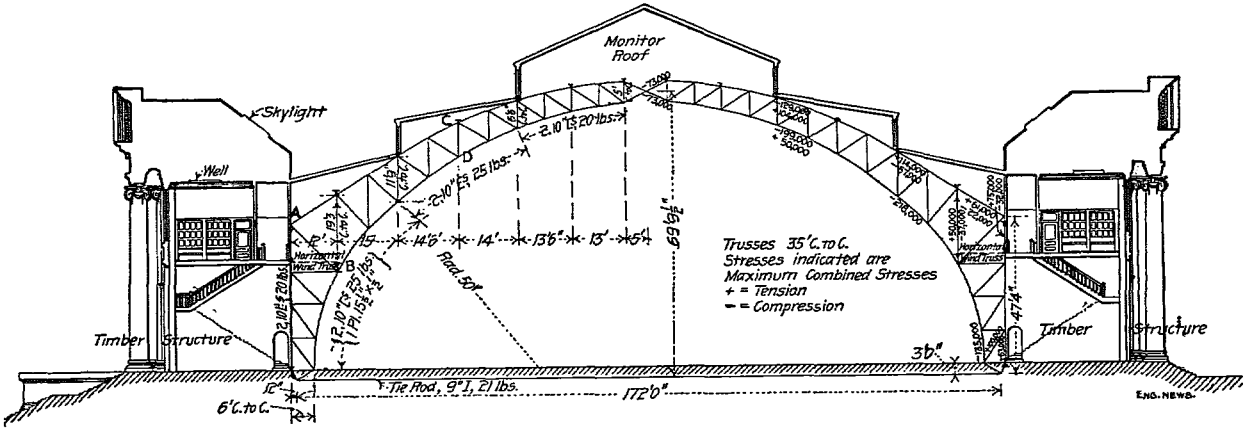


FIG. 183. CROSS SECTION OF GOVERNMENT BUILDING, ST. LOUIS EXPOSITION.

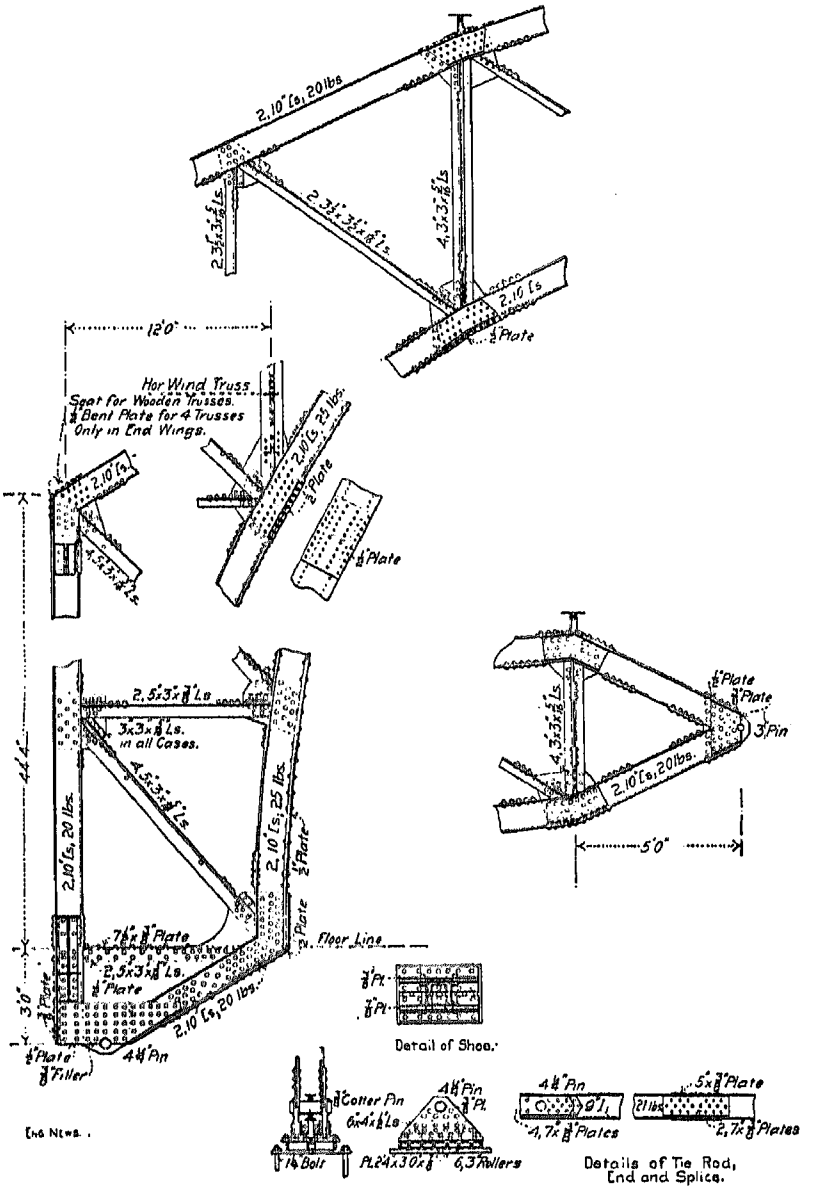


FIG. 184. DETAILS OF ARCH.

Reinforced Concrete Round-house for the Canadian Pacific Railway at Moose Jaw, Canada.*

The round-house is annular in plan and occupies a half circle, the diameter of which is 350′ 5″. The radial width of the house is 80 feet, and it has 22 stalls divided into two groups of 11 stalls each, by a radial fire wall. Each stall subtends an outside wall space of 25′.

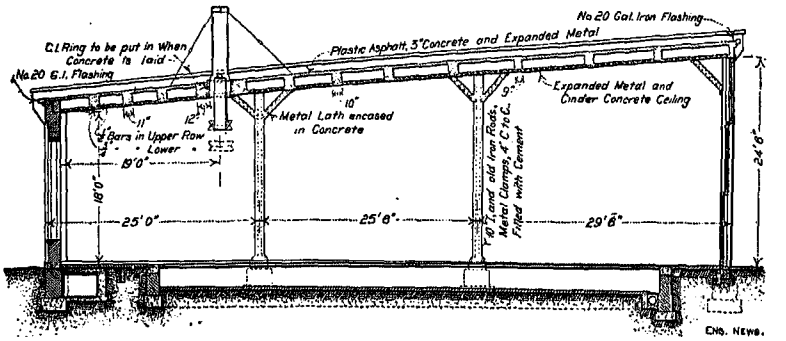


Fig. 185. Cross Section of Reinforced Concrete Round-house.

The end walls, fire wall and outside wall are of plain concrete, the latter being enlarged by buttresses or pilasters between stall spaces to carry the outer ends of the radial roof girders, which are further supported by an outside wall column, and two intermediate columns. The combined roof, wall and column construction is shown in Fig. 185.

Each radial girder consists of an 18″ @ 55 lbs. I-beam encased in concrete as shown in Fig. 186, and is supported by an outside wall pier and three I-beam columns enclosed in concrete as shown in Fig. 187. The space between the main roof girders is spanned by reinforced concrete beams, carrying a reinforced roof slab and a suspended

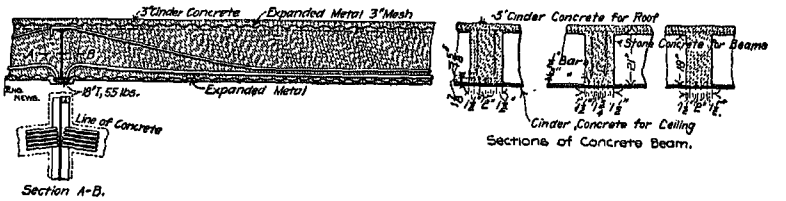
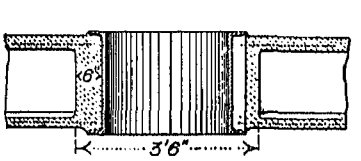
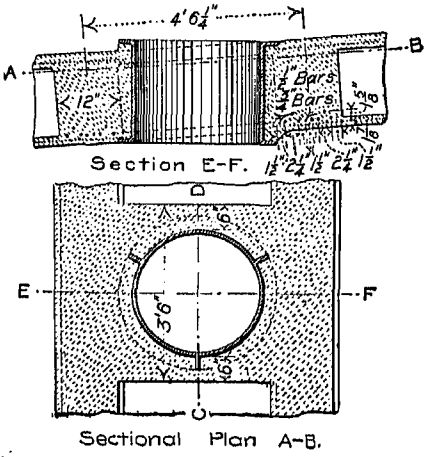


Fig. 186. Cross Section Showing Construction of Roof Slabs.

* Engineering News, February 9, 1905.





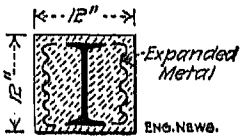


FIG. 187. I-BEAM COLUMN.

FIG. 188. HOLES IN ROOF FOR SMOKE JACK.

metal lath and plaster ceiling as shown in Fig. 186. The main girders and the angles which knee-brace them to the columns are like the columns, encased in concrete. There is, therefore, no unprotected structural metal in the round-house.

The reinforced concrete beams shown in Fig. 186 are of uniform depth, but vary in width and amount of reinforcement with the span and are composed of 1–3–5 Portland cement, gravel concrete.

The beam reinforcement consists of plain rods attached to the beams as shown in Fig. 186. The roof slabs are composed of cement and washed cinders, and the ceiling slab consists simply of expanded metal wired to the beams and given two coats of cement plaster, the first coat containing enough lime to make it work well under the trowel, and the second coat being a 1 cement to 1 sand mortar.

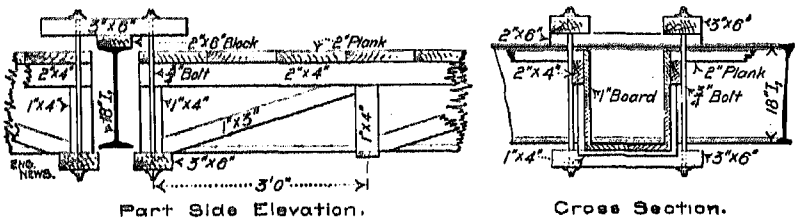


FIG. 189. FORMS FOR REINFORCED CONCRETE ROOF.

It should be noted that no joints have been provided in the roof to accommodate expansion and contraction. No cracks appeared during the first year, and none are expected to appear.

The roof is reinforced where the metal smoke jacks are located, as shown in Fig. 185 and Fig. 188.

The forms used in constructing the roof are shown in Fig. 189. The round-house is standard for the Canadian Pacific Railway, and cost $3,000 per stall.

APPENDIX I.

GENERAL SPECIFICATIONS FOR

STEEL FRAME MILL BUILDINGS

MILO S. KETCHUM, ASSOC. M. AM. SOC. C. E.

1903

GENERAL DESCRIPTION

Height of Building.

1. The height of the building shall be the distance from the top of the masonry to the under side of the bottom chord of the truss.

Dimensions of Building.

2. The width and length of the building shall be the extreme distance out to out of framing or sheathing.

Length of Span.

3. The length of trusses and girders in calculating stresses shall be considered as the distance from center to center of end bearings when supported, and from end to end when fastened between columns by connection angles.

Pitch of Roof.

4. The pitch of roof for corrugated steel shall preferably be not less than ¼ (6″ in 12″), and in no case less than ⅕. For a pitch less than ⅕ some other covering than corrugated steel shall be used.

Spacing of Trusses.

5. Trusses shall be spaced so that simple shapes may be used for purlins. The spacing should be about 16 feet for spans of, say, 50 feet and about 20 to 22 feet for spans of, say, 100 feet. For longer spans than 100 feet the purlins may be trussed and the spacing may be increased.

Spacing of Purlins.

6. Purlins shall be spaced not to exceed 4′ 9″ where corrugated steel is used, and shall be placed at panel points of the trusses.

Form of Trusses.

7. The trusses shall preferably be of the Fink type with panels so subdivided that panel points will come under the purlins. If it is not practicable to place the purlins at panel points, the upper chords of the trusses shall be designed to take both the flexural and direct stresses. Trusses shall preferably be riveted trusses.

Bracing.

8. Bracing in the plane of the lower chord shall be stiff; bracing in the planes of the top chords, the sides and the ends may be made adjustable.

Proposals.

9. Contractors in submitting proposals shall furnish complete stress sheets, general plans of the proposed structures giving sizes of material, and such detail plans as will clearly show the dimensions of the parts, modes of construction and sectional areas.

Detail Plans.

10. The successful contractor shall furnish all working drawings required by the engineer free of cost. Working drawings will, as far as possible, be made on standard size sheets 24″ × 36″ out to out, 22″ × 34″ inside the inner border lines.

Approval of Plans.

11. No work shall be commenced or materials ordered until the working drawings are approved in writing by the engineer. The contractor shall be responsible for dimensions and details on the working plans, and the approval of the detail plans by the engineer will not relieve the contractor of this responsibility.

LOADS.

12. The trusses shall be designed to carry the following loads:

Dead Loads.

13. **DEAD LOADS. — Weight of Trusses** — The weight of trusses per square foot of horizontal projection, up to 150 feet span shall be calculated by the formula

$$W = \frac{P}{45}\left(1 + \frac{L}{5\sqrt{A}}\right)$$

where W = weight of trusses per square foot of horizontal projection;

P = capacity of truss in pounds per square foot of horizontal projection;

L = span of the truss in feet;

A = distance between trusses in feet.

Corrugated Steel.

14. **Weight of Covering.**—The weight of corrugated steel shall be taken from Table I.

TABLE I.

WEIGHT OF FLAT, AND CORRUGATED STEEL SHEETS WITH 2½-INCH CORRUGATIONS

Gage No.	Thickness in inches	Weight per Square (100 sq. ft.)			
		Flat Sheets		Corrugated Sheets	
		Black	Galvanized	Black Painted	Galvanized
16	.0625	250	266	275	291
18	.0500	200	216	220	236
20	.0375	150	166	165	182
22	.0313	125	141	138	154
24	.0250	100	116	111	127
26	.0188	75	91	84	99
28	.0156	63	79	69	86

When two corrugations side lap and six inches end lap are used add 25 per cent to the above weights; when one corrugation side lap and four inches end lap are used add 15 per cent to the above weights to obtain weight of corrugated steel laid. For paint add 2 pounds per square. The weight of covering shall be reduced to weight per square foot of horizontal projection before combining with weight of trusses.

Slate.

15. Slate laid with 3-inch lap shall be taken at a weight of 7½ pounds per square foot of inclined roof surface for 3-16″ slate 6″ × 12″, and 6½ pounds per square foot of inclined roof surface for 3-16″ slate 12″ × 24″, and proportionately for other sizes.

Tile.

16. Terra-cotta tile roofing weighs about 6 pounds per square foot for tile 1 inch thick; the actual weight of tile and other roof coverings not named shall be used.

Sheathing and Purlins.

17. Sheathing of dry pine lumber shall be assumed to weigh 3 pounds per foot and dry oak purlins 4 pounds per foot board measure.

Miscellaneous Loads.

18. The exact weight of sheathing, purlins, bracing, ventilators, cranes, etc., shall be calculated.

Snow Loads.

19. **SNOW LOADS.**—Snow loads shall be taken from the diagram in Fig. 1.

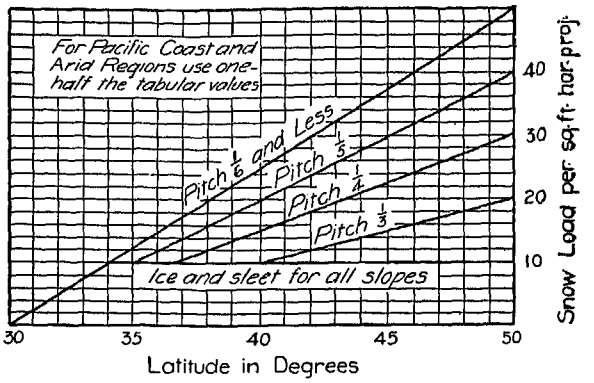


FIG. 1. SNOW LOAD ON ROOFS FOR DIFFERENT LATITUDES, IN LBS. PER SQUARE FOOT.

Wind Loads.

20. **WIND LOADS.**—The normal wind pressure on trusses shall be computed by Duchemin's formula, Fig. 2, with $P = 30$ pounds per square foot, except for buildings in exposed locations, where $P = 40$ pounds per square foot shall be used.

21. The sides and ends of buildings shall be computed for a normal wind load of 20 pounds per square foot of exposed surface for buildings 30 feet and less to the eaves; 30 pounds per square foot of exposed surface for buildings 60 feet to the eaves, and in proportion for intermediate heights.

Mine Buildings.

22. Mine, smelter and other buildings exposed to the action of corrosive gases shall have their dead loads increased 25 per cent.

Concentrated Loads.

23. Concentrated loads and crane girders shall be considered in determining dead loads.

Purlins.

24. Purlins shall be designed for a normal load of not less than 30 lbs. per square foot.

Roof Covering.

25. Roof covering shall be designed for a normal load of not less than 30 lbs. per square foot.

Minimum Loads.

26. No roof shall, however, be designed for an equivalent load of less than 30 pounds per square foot of horizontal projection.

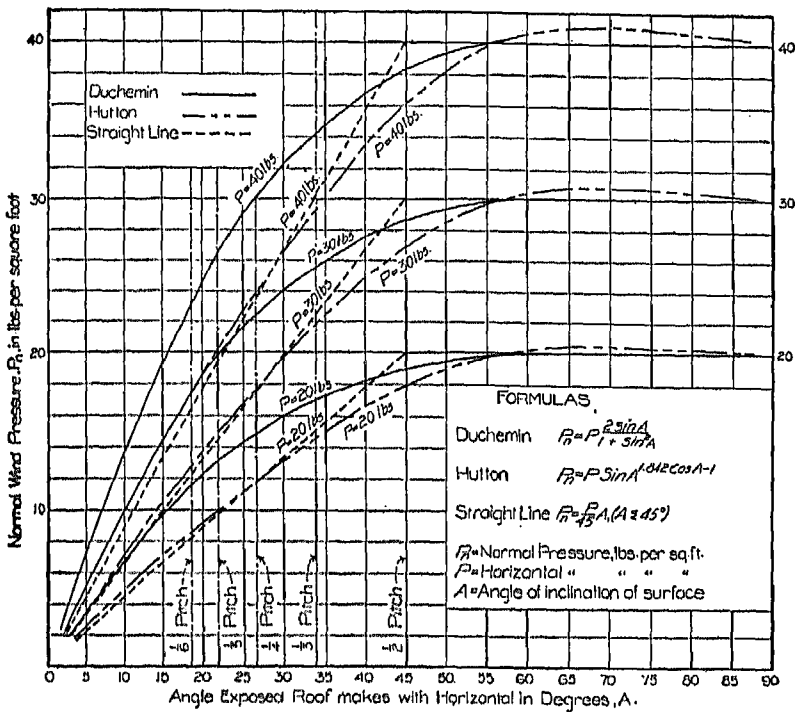


FIG. 2. NORMAL WIND LOAD ON ROOF ACCORDING TO DIFFERENT FORMULAS.

PROPORTION OF PARTS.

27. In proportioning the different parts of the structure the maximum stresses due to the combinations of the dead and wind load; dead and snow load; or dead, minimum snow and wind load are to be provided for. Concentrated loads where they occur must be provided for. **Stresses.**

28. Allowable Unit Tensile Stresses for Medium Steel. **Tensile Stress.**

	Pounds per square inch
Shapes, main members, net section	16,000
Bars	16,000
Bottom flanges of rolled beams	16,000
Shapes, laterals, net section	20,000
Iron rods for laterals	20,000
Plate girder webs, shearing on net section	10,000

Shapes liable to sudden loading as when used for crane girders.....10,000

Expansion rollers per lineal inch.....$600 \times D$

where D = diameter of roller in inches.

Laterals shall be designed for the maximum stresses due to 5,000 pounds initial tension and the maximum stress due to wind.

Compressive Stress.

29. Allowable Unit Compressive Stress for Medium Steel. For direct dead, snow and wind loads

$$S = 16{,}000 - 70\frac{l}{r}$$

where

S = allowable unit stress in pounds per square inch;

l = length of member in inches c. to c. of end connections;

r = least radius of gyration of the member in inches.

Plate Girders.

30. Top flanges of plate girders shall have the same gross area as the tension flanges.

31. Shear in webs of plate girders shall not exceed 10,000 pounds per square inch.

Alternate Stress.

32. Members and connections subject to alternate stresses shall be designed to take each kind of stress.

Combined Stress.

33. Members subject to combined direct and bending stresses shall be proportioned according to the following formula:

$$S = \frac{P}{A} + \frac{M y_1}{I \pm \frac{P l^2}{10 E}}$$

where

S = stress per square inch in extreme fibre;

P = direct load;

A = area of member;

M = bending moment in inch-pounds;

y_1 = distance from neutral axis to extreme fibre;

I = moment of inertia of member;

l = length member, or distance from point of zer moment to end of member in inches;

E = modulus of elasticity = 28,000,000.

When combined direct and flexural stress due to wind is considered add 25 per cent to the above allowable tensile and compressive stresses.

Soft Steel.

34. Soft steel may be used in mill buildings with unit stresses ten per cent less than those allowed for medium steel.

Stress due to Weight of Member.

35. Where the stress due to the weight of the member or due to an eccentric load exceeds the allowable stress for direct loads by more than ten per cent, the section shall be increased until the total stress does not exceed the above allowable stress for direct loads by more than ten per cent.

The eccentric stress caused by connecting angles by one leg when used as ties or struts shall be calculated, or only one leg will be considered effective.

Rivets.

36. Rivets shall be so spaced that the shearing stress shall not exceed 11,000 pounds per square inch; nor the pressure on the bearing surface (diameter × thickness of piece) of the rivet hole exceed 22,000 pounds per square inch.

Rivets in lateral connections may have stresses 25 per cent in excess of the above.

Field rivets shall be spaced for stresses two-thirds those allowed for shop rivets.

Field bolts, when allowed, shall be spaced for stresses two-thirds those allowed for field rivets.

Rivets and bolts must not be used in direct tension.

Pins.

37. Pins shall be proportioned so that the shearing stress shall not exceed 11,000 pounds per square inch; nor the pressure on the bearing surface (diameter × thickness of piece) of the pin hole exceed 22,000 pounds per square inch; nor the extreme fibre stress due to cross bending exceed 24,000 pounds per square inch when the applied forces are assumed as acting at the center of the members.

Rolled Beams.

38. Rolled beams shall be proportioned by their moments of inertia.

Plate Girders.

39. Plate girders shall be proportioned on the assumption that the flanges take all the bending moment, and that the shear is resisted by the web.

The distance between centers of gravity of the flange areas shall be considered as the effective depth of all girders.

Stiffeners.

40. The webs of plate girders shall be stiffened at bearings and at all points of concentrated loading, and at inter-

mediate points, wherever the shearing per square inch exceeds the stress allowed by the following formula:

Allowed shearing stress $= 12{,}500 - 90\, d \div t$

where d = depth, and t = thickness of web plate.

41. Compression flanges of plate girders shall be stayed transversely when their length is more than thirty times their width.

Timber.

42. The allowable stresses in timber purlins and other timbers shall be taken from Table II.

TABLE II.

ALLOWABLE WORKING UNIT STRESSES, IN POUNDS, PER SQUARE INCH.

Kind of Timber.	Tension.		Compression.			Transverse.		Shearing.	
			With Grain.						
	1	2	3	4	5	6	7	8	9
	With Grain.	Across Grain.	End Bearing.	Columns Under 15 Diams.	Across Grain.	Extreme Fibre Stress.	Modulus of Elasticity.	With Grain.	Across Grain.
Factor of Safety.	Ten.	Ten.	Five.	Five.	Four.	Six.	Two.	Four.	Four
White Oak	1000	200	1400	960	500	1000	550000	200	1000
White Pine	700	50	1100	700	200	700	500000	100	500
South. Long-leaf or Georgia Yellow Pine	1200	60	1600	1000	350	1200	850000	150	1250
Douglas. Oregon and Yellow Fir	1200	.....	1600	1200	300	1100	700000	150	.....
Washington Fir or Pine (Red Fir)	1000	.....	.....	.....	.....	800	.....	.....	.....
Northern or Short-leaf Yellow Pine	900	50	1200	800	250	1000	600000	100	1000
Red Pine	900	50	1200	800	200	800	600000	.....	.....
Norway Pine	800	.....	1200	800	200	700	600000	.....	.....
Canadian (Ottawa) White Pine	1000	.....	.....	1000	.....	.....	.....	100	.....
Canadian (Ontario) Red Pine	1000	.....	.....	1000	.....	800	700000	100	.....
Spruce and Eastern Fir	800	50	1200	800	200	700	600000	100	750
Hemlock	600	.....	.....	800	150	600	400000	100	600
Cypress	600	.....	1200	800	200	800	450000	.....	.....
Cedar	800	.....	1200	800	200	800	350000	.....	400
Chestnut	900	.....	.....	1000	250	800	500000	150	400
California Redwood	700	.....	.....	800	200	750	300000	100	.....
California Spruce	.....	.....	.....	800	.....	800	600000	.....	.....

The allowable stress, P, in timber columns longer than 15 diameters shall be obtained by means of the formula

$$P = C - \frac{C}{100}\,\frac{l}{d}$$

where C = value in column 4 above; l = length of column and d least width of column, both in feet.

COVERING.

Corrugated Steel.

43. Corrugated steel shall generally have 2½ inch corrugations when used for roof and sides of buildings, and 1¼ inch corrugations when used for lining buildings. The mini-

mum gage of corrugated steel shall be No. 22 for roofs, No. 24 for sides and No. 26 for lining.

The gage of corrugated steel in U. S. Standard Gage, and weight per square foot shall be shown on the general plan.

Spacing Purlins.

44. The span, or center to center distance of purlins, shall not exceed the distance given in Fig. 3 for a safe load of 30 pounds per square foot. Corrugated steel sheets shall preferably span two purlin spaces.

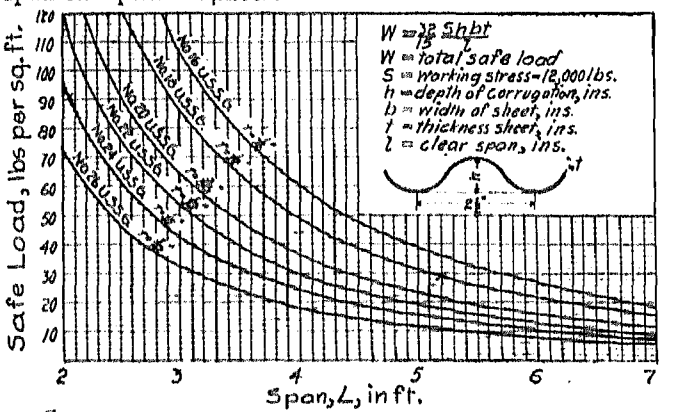


FIG. 3. SAFE UNIFORM LOAD IN POUNDS FOR CORRUGATED STEEL FOR DIFFERENT SPANS IN FEET.

End and Side Laps.

45. Corrugated steel shall be laid with two corrugations side lap and six inches end lap when used for roofing, and one corrugation side lap and four inches end lap when used for siding.

Fastening.

46. Corrugated steel shall be fastened to the purlins by means of galvanized iron straps ¾ inches wide by No. 18 gage, spaced 8 to 12 inches apart; by clinch nails spaced 8 to 12 inches apart; or by nailing directly to spiking strips with 8d barbed nails, spaced 8 inches apart. Spiking strips shall preferably be used with anti-condensation lining. Bolts, nails and rivets shall always pass through the top of corrugations. Side laps shall be riveted with copper or galvanized iron rivets 8 to 12 inches apart on the roof and 1½ to 2 feet apart on the sides.

Corrugated Steel Lining.

47. Corrugated steel lining on the sides shall be laid with one corrugation side lap and four inches end lap. Girts for corrugated steel lining shall be spaced for a safe load of 25 pounds per square foot as given in Fig. 3.

Anti-Condensation Lining.

48. Anti-condensation roof lining shall preferably be used to prevent dripping in engine houses and similar buildings and shall be constructed as follows: Galvanized wire poultry netting is fastened to one eave purlin and is passed over the ridge, stretched tight and fastened to the other eave purlin. The edges of the wire are woven together, and the netting is fastened to the spiking strips, where used, by means of small staples. On the netting are laid two layers of asbestos paper 1-16-inch thick, and two layers of tar paper. The corrugated steel is then fastened to the purlins in the usual way. Three-sixteenth-inch stove bolts with 1″ × ⅛″ × 4″ plate washers on the lower side are used for fastening the side laps together and for supporting the lining; or the purlins may be spaced one-half the usual distance where anti-condensation lining is used, and the stove bolts omitted.

Flashing.

49. Valleys or corners around stacks shall have flashing extending at least 12 inches above where water will stand, and shall be riveted or soldered if necessary, to prevent leakage.

Flashing shall be provided above doors and windows.

Ridge Roll.

50. All ridges shall have a ridge roll securely fastened to the corrugated steel.

Corner Finish.

51. All corners shall be covered with standard corner finish securely fastened to the corrugated steel.

Cornice.

52. At the gable ends the corrugated steel on the roof shall be securely fastened to a finish angle or channel connected to the end of the purlins or, where molded cornices are used, to a piece of timber fastened to the ends of the purlins.

Gutters.

53. Gutters and conductors shall be furnished at least equal to the requirements of the following table:

Span of Roof	Gutter	Conductor
Up to 50′	6″	4″ every 40′
50′ to 70′	7″	5″ " "
70′ to 100′	8″	5″ " "

Gutters shall have a slope of at least 1″ in 15′. Gutters and conductors shall be made of galvanized steel not lighter than No. 24.

Ventilators.

54. Ventilators shall be provided and located so as to properly ventilate the building. They shall have a net opening for each 100 square feet of floor space of not less than one-

fourth square foot for clean machine shops and similar buildings; of not less than one square foot for dirty machine shops; of not less than four square feet for mills; and not less than six square feet for forge shops, foundries and smelters.

Shutters and Louvres.

55. Openings in ventilators shall be provided with shutters, sash, or louvres, or may be left open as specified.

Shutters must be provided with a satisfactory device for opening and closing.

Louvres must be designed to prevent the blowing in of rain and snow, and must be made stiff so that no appreciable sagging will occur. They shall be made of not less than No. 20 gage galvanized steel for flat louvres and No. 24 gage galvanized steel for corrugated louvres.

Circular Ventilators.

56. Circular ventilators when used must be designed so as to prevent down drafts. Net opening only shall be used in calculations.

Windows.

57. Windows shall be provided in the exterior walls equal to not less than 10 per cent of the entire exterior surface in mill buildings, and of not less than 25 per cent in machine shops, factories and similar buildings.

Window glass up to 12" × 14" may be single strength, over 12" × 14" the glass shall be double strength. Window glass shall be A grade except in smelters, foundries, forge shops, and similar structures where it may be B grade. The sash and frames shall be constructed of white pine.

Skylights.

58. At least half of the lighting shall preferably be by means of skylights, or sash in the sides of ventilators.

Skylights shall be glazed with wire glass, or wire netting shall be stretched beneath the skylights to prevent the broken glass from falling into the building. Where there is danger of the skylight glass being broken by objects falling on it, a wire netting guard shall be provided on the outside.

Skylight glass shall be carefully set, special care being used to prevent leakage. Leakage and condensation on the inner surface of the glass shall be carried to the down-spouts, or outside the building by condensation gutters.

59. Windows in sides of buildings shall be made with counterbalanced sash, and in ventilators shall be made with sliding or swing sash.

Doors.

60. Doors are to be furnished as specified and are to be provided with hinges, tracks, locks and bolts. Single doors up to 4 feet and double doors up to 8 feet shall preferably be swung on hinges; large doors, double and single, shall be arranged to slide on overhead tracks, or may be counterbalanced to lift up between vertical guides.

Steel doors shall be firmly braced and shall be covered with No. 24 corrugated steel with 1¼-inch corrugations.

The frames of sandwich doors shall be made of two layers of ⅞-in. matched white pine, placed diagonally, and firmly nailed with clinch nails. The frame shall be covered on each side with a layer of No. 26 corrugated steel with 1¼-inch corrugations.

DETAILS OF CONSTRUCTION.

Details.

61. All connections and details shall be of sufficient strength to develop the full strength of the member.

Pitch of Rivets.

62. The pitch of rivets shall not exceed 6 inches, or sixteen times the thickness of the thinnest outside plate in the line of stress, nor forty times the thickness of the thinnest outside plate at right angles to the line of stress. The pitch shall never be less than three diameters of rivet. At the ends of compression members the pitch shall not exceed four diameters of the rivet for a length equal to twice the width of the member.

63. The distance between the edge of any piece and the center of a rivet hole must never be less than 1¼ inches, except in angles having a 2″ leg, where it shall be 1⅛ inches.

Diameter of Punch.

64. The diameter of the punch shall not exceed the diameter of the rivet; nor the diameter of the die exceed the diameter of the punch by more than 1-16 inch.

Net Sections.

65. The effective diameter of a driven rivet will be assumed the same as its diameter before driving. In deducting the rivet holes to obtain net sections in tension members, the diameter of the rivet-holes will be assumed as ⅛ inch larger than the undriven rivet.

Minimum Sections

66. No metal of less thickness than ¼ inch shall be used except for fillers; and no angles less than 2″ × 2″ × ¼″. No upset rod shall be less than ⅝ inch in diameter. Sag rods may be as small as ⅜-inch.

Upset Rods.

67. All rods with screw ends except sag rods must be upset at the ends so that the diameter at the base of the threads shall be 1-16 inch larger than any part of the body of the bar.

Upper Chords.

68. Upper chords shall have symmetrical cross-sections, and shall preferably consist of two angles back to back.

Compression Members.

69. All other compression members except sub-struts shall be composed of sections symmetrically placed. Sub-struts may consist of a single section.

Columns.

70. Side posts which take flexure shall preferably be composed of 4 angles laced, or 4 angles and a plate. Where side posts do not take flexure and carry heavy loads they shall preferably be composed of two channels laced.

Posts in end framing shall preferably be composed of I beams or 4 angles laced. Corner columns shall preferably be composed of one angle.

Crane Posts.

The cross-bending stress due to eccentric loading in columns carrying cranes shall be calculated.

71. Laced compression members shall be stayed at the ends by batten plates placed as near the end of the member as practicable and having a length not less than the greatest width of the member. The thickness of batten plates must not be less than 1-40 of the distance between rivet lines at right angles to axis of member.

Lacing.

72. Single lattice bars shall have a thickness of not less than 1-40, and double bars connected by a rivet at the intersection of not less than 1-60 of the distance between the rivets connecting them to the member; they shall make an angle not less than 45° with the axis of the member; their width shall be in accordance with the following standards, generally:

SIZE OF MEMBER.	WIDTH OF LACING BARS.
For 15-inch channels, or built sections with 3½ and 4-inch angles.	2½ inches (⅞-inch rivets).
For 12, 10 and 9-inch channels, or built sections with 3-inch angles.	2¼ inches (¾-inch rivets).

For 8 and 7-inch channels, or built sections with 2½-inch angles. } 2 inches (⅝-inch rivets).

For 6 and 5-inch channels, or built sections with 2-inch angles. } 1¾ inches (½-inch rivets).

Where laced members are subjected to bending, the size of lacing bars or angles shall be calculated or a solid web plate shall be used.

Pin Plates. 73. All pin holes shall be reenforced by additional material when necessary, so as not to exceed the allowable pressure on the pins. These reenforcing plates must contain enough rivets to transfer the proportion of pressure which comes upon them, and at least one plate on each side shall extend not less than 6 inches beyond the edge of the tie plate.

Maximum Length. 74. No compression member shall have a length exceeding 125 times its least radius of gyration for main members, nor 150 times its least radius of gyration for laterals and sub-members.

Splices. 75. In compression members joints with abutting faces planed shall be placed as near the panel points as possible, and must be spliced on all sides with at least two rows of rivets on each side of the joint. Joints with abutting faces not planed must be fully spliced.

Splices. 76. Joints in tension members shall be fully spliced.

Tension Members. 77. Tension members shall preferably be composed of angles or shapes capable of taking compression as well as tension. Flats riveted at the ends shall not be used.

78. Main tension members shall preferably be made of 2 angles, 2 angles and a plate, or 2 channels laced. Secondary tension members may be made of a single shape.

Eye-Bars. 79. Heads of eye-bars shall be so proportioned as to develop the full strength of the bar. The heads shall be forged and not welded.

Pins. 80. Pins must be turned true to size and straight, and must be driven to place by means of pilot nuts.

The diameter of pin shall not be less than ¾ of the depth of the widest bar attached to it.

The several members attached to a pin shall be packed so as to produce the least bending moment on the pin, and all vacant spaces must be filled with steel fillers.

Rods.

81. Long laterals may be made of rods with clevis or sleeve nut adjustment. Bent loops shall not be used.

Spacing Trusses.

82. Trusses shall preferably be spaced so as to allow the use of single pieces of rolled sections for purlins. Trussed purlins shall be avoided if possible.

Purlins and Girts.

83. Purlins and girts shall preferably be composed of single sections—channels, angles or Z-bars placed with web at right angles to the trusses and posts and legs turned down.

Fastening.

84. Purlins and girts shall be attached to the top chord of trusses and to columns by means of angle clips with two rivets in each leg.

Spacing.

85. Purlins shall be spaced at distances apart not to exceed the span as given for a safe load of 30 pounds, and girts for a safe load of 25 pounds in Fig. 3.

Timber Purlins.

86. Timber purlins shall be attached and spaced the same as steel purlins.

Base Plates.

87. Base plates shall never be less than ½ inch in thickness, and shall be of sufficient thickness and size so that the pressure on the masonry shall not exceed 250 pounds per square inch.

Anchors.

88. Columns shall be anchored to the foundations by means of two anchor bolts not less than 1″ in diameter upset, placed as wide apart as practicable in the plane of the wind. The anchorage shall be calculated to resist the bending moment at the base of the columns.

Lateral Bracing.

89. Lateral bracing shall be provided in the plane of the top and bottom chords, side and ends; knee braces in the transverse bents; and sway bracing wherever necessary. Lateral bracing shall be designed for an initial stress of 5,000 pounds in each member, and provision must be made for putting this initial stress into the members in erecting.

Temperature.

90. Variations in temperature to the extent of 150 degrees F. shall be provided for.

WORKMANSHIP.

Workmanship.

91. All workmanship shall be first-class in every particular. Due regard must be had for the neat and attractive appearance of the finished structure, and details of an unsightly character will not be allowed.

Riveted Work Punching.

92. All riveted work shall be punched accurately with holes 1-16 of an inch larger than the size of the rivet, and when the pieces forming one built member are put together, the holes must be truly opposite; no drifting to distort the metal will be allowed; if the hole must be enlarged to admit the rivet, it must be reamed.

Holes for *Field Rivets.*

93. All holes for field rivets in splices in tension members shall be accurately drilled to an iron templet or reamed while the connecting parts are temporarily put together.

Planing and Reaming.

94. In medium steel over $\frac{5}{8}$ of an inch thick, all sheared edges shall be planed, and all holes shall be drilled or reamed to a diameter of $\frac{1}{8}$ of an inch larger than the punched holes, so as to remove all the sheared surface of the metal. Steel which does not satisfy the drifting test must have holes drilled.

Rivets.

95. The rivet heads must be of approved hemispherical shape, and of a uniform size for the same size rivets throughout the work. They must be full and neatly finished throughout the work and concentric with the rivet hole.

96. All rivets when driven must completely fill the holes, the heads be in full contact with the surface, or countersunk when so required.

Riveters.

97. Rivets shall be machine driven wherever possible. Power riveters shall be direct-acting machines, worked by steam, hydraulic pressure, or compressed air.

Bolts.

98. When members are connected by bolts which transmit shearing strains, such bolts must have a driving fit.

Neat Finish.

99. The several pieces forming one built member must fit closely together, and when riveted shall be free from twists, bends, or open joints.

100. All portions of the work exposed to view shall be neatly finished.

Contact Surfaces.

101. All surfaces in contact shall be painted before they are put together.

Forged Work Eye-Bars.

102. The heads of eye-bars shall be made by upsetting, rolling, or forging into shape. Welds in the body of the bar will not be allowed.

103. The bars must be perfectly straight before boring.

104. The holes shall be in the center of the head and on the center line of the bar.

105. All eye-bars shall be annealed.

Machine Work Facing.

106. All abutting surfaces in compression members shall be truly faced to even bearings, so that they shall be in such contact throughout as may be obtained by such means.

Pin Holes.

107. Pin holes shall be bored truly parallel with one another and at right angles to the axis of the member unless otherwise shown in drawings; and in pieces not adjustable for length, no variation of more than 1-64 of an inch for every 20 feet will be allowed in the length between centers of pin holes.

108. Bars which are to be placed side by side in the structure shall be bored at the same temperature, and shall be of such equal length that, upon being piled on each other, the pins shall pass through the holes at both ends at the same time without driving.

Pins.

109. All pins shall be accurately turned to a gage, and shall be straight and smooth.

Play in Pin Holes.

110. The clearance between pin and pin hole shall be 1-50 of an inch for pins up to 3¼ inches in diameter, and 1-32 for larger pins.

Pilot Nuts.

111. All pins shall be supplied with steel pilot nuts, for use during erection.

QUALITY OF MATERIAL.

STEEL.

Process of Manufacture.

112. All steel must be made by the open hearth process, and if by acid process, shall contain not more than 0.08 per cent of phosphorus, and if by basic process, not more than 0.06 per cent of phosphorus, nor more than 0.05 per cent of sulphur, and must be uniform in character for each specified kind.

Finish.

113. The finished bars, plates and shapes must be free from injurious seams, flaws, or cracks, and have a clean smooth finish.

No work shall be done on any steel between the temperature of boiling water and of ignition of hard wood saw dust.

Test Pieces.

114. The tensile strength, limit of elasticity and ductility, shall be determined from a standard test-piece, cut from the finished material, of at least ½ square inch section. All broken samples must show a silky fracture of uniform color.

Annealed Test Pieces.

115. Material which is to be used without annealing or further treatment is to be tested in the condition in which it comes from the rolls. When material is to be annealed or otherwise treated before use, the specimen representing such material is to be similarly treated before testing.

Marking.

116. Every finished piece of steel shall be stamped with the blow number identifying the melt.

Physical Properties.

117. Steel shall be of three grades: Rivet, Soft and Medium.

Rivet Steel.

118. Rivet steel shall have: Ultimate strength, 50,000 to 58,000 pounds per square inch. Elastic limit, not less than one-half the ultimate strength. Elongation, 26 per cent in 8 inches. Bending test, after or before heating to a light cherry red and cooling in water, 180 degrees flat on itself, without fracture on outside of bent portion.

Soft Steel.

119. Soft steel shall have: Ultimate strength, 54,000 to 62,000 pounds per square inch. Elastic limit, not less than one-half the ultimate strength. Elongation, 25 per cent in 8 inches. Bending test, after or before heating to a light cherry red and cooling in water, 180 degrees flat on itself, without fracture on outside of bent portion.

Medium Steel.

120. Medium steel shall have: Ultimate strength, 60,000 to 68,000 pounds per square inch. Elastic limit, not less than one-half the ultimate strength. Elongation, 22 per cent in 8 inches. Bending test, 180 degrees to a diameter equal to thickness of piece tested, without fracture on outside of bent portion.

Full Size Test of Steel Eye-Bars.

121. Full size test of steel eye-bars shall be required to show not less than 10 per cent elongation in the body of the bar, and tensile strength not more than 5,000 pounds below the minimum tensile strength required in specimen tests of the grade of steel from which they are rolled. The bars will be required to break in the body, but should a bar break in the

head, but develop 10 per cent elongation and the ultimate strength specified, it shall not be cause for rejection, provided not more than one-third of the total number of bars tested break in the head; otherwise the entire lot will be rejected.

Pin Steel.

122. Pins made of either of the above mentioned grades of steel shall, on specimen test pieces cut from finished material, fill the requirements of the grade of steel from which they are rolled, excepting the elongation, which shall be decreased 5 per cent from that specified.

Drifting.

123. In steel $\frac{5}{8}$ inch or less in thickness punched rivet holes, pitched two diameters from a sheared edge, must stand drifting until the diameter is one-third larger than the original hole, without cracking the metal.

Slabs for Plates.

124. The slabs for rolling plates shall be rolled from ingots of at least twice their cross-section.

Pins.

125. Pins up to 7 inches diameter shall be rolled.

Variation in Weight.

126. A variation in cross-section or weight of rolled material of more than $2\frac{1}{2}$ per cent from that specified, may be cause for rejection, except in the case of plates which will be covered by the manufacturer's standard specifications (Cambria Steel, page 345).

STEEL CASTINGS.

Steel Castings.

127. Steel castings shall be made of open hearth steel containing from 0.25 to 0.40 per cent carbon, and not over 0.08 per cent of phosphorus nor 0.05 per cent sulphur, and shall be practically free from blow holes.

CAST IRON.

Cast Iron.

128. Except where chilled iron is specified, all castings shall be of tough, gray iron, free from injurious cold shuts or blow holes, true to pattern, and of workmanlike finish. Test bars one inch square, loaded in middle between supports 12 inches apart, shall bear 2,500 pounds or over, and deflect 0.15 of an inch before rupture.

WROUGHT IRON.

Character and Finish.

129. All wrought iron must be tough, ductile, fibrous and of uniform quality. Finished bars must be thoroughly welded

during the rolling, and be straight, smooth and free from injurious seams, blisters, buckles, cracks or imperfect edges.

Manufacture.

130. No specific process or provision of manufacture will be demanded, provided the material fulfills the requirements of these specifications.

Standard Test Piece

131. The tensile strength, limit of elasticity and ductility, shall be determined from a standard test piece of as near ½ square inch sectional area as possible. The elongation shall be measured on an original length of 8 inches.

Elastic Limit.

132. Iron of all grades shall have an elastic limit of not less than 26,000 pounds per square inch.

Tension.

133. When tested in specimens of uniform sectional area of at least ½ square inch the iron shall show a minimum ultimate strength of 50,000 pounds per square inch, and a minimum elongation of 18 per cent in 8 inches.

Bending Test.

134. All iron for tension members must bend cold through 90 degrees to a curve whose diameter is not over twice the thickness of the piece, without cracking. When nicked on one side and bent by a blow from a sledge, the fracture must be mostly fibrous.

TIMBER.

Timber.

135. The timber shall be strictly first-class spruce, white pine, Douglas fir, Southern yellow pine, or white oak timber; sawed true and out of wind, full size, free from wind shakes, large or loose knots, decayed or sapwood, wormholes or other defects impairing its strength or durability.

PAINTING.

Painting

136. All iron work before leaving the shop shall be thoroughly cleaned from all loose scale and rust, and be given one good coating of pure boiled linseed oil or paint as specified, well worked into all joints and open spaces.

137. In riveted work, the surfaces coming in contact shall each be painted before being riveted together.

138. Pieces and parts which are not accessible for painting after erection shall have two coats of paint.

139. The paint shall be a good quality of red lead or graphite paint, ground with pure linseed oil, or such paint as may be specified in the contract.

140. After the structure is erected, the iron work shall be thoroughly and evenly painted with two additional coats of paint, mixed with pure linseed oil, of such quality and color as may be selected. Painting shall be done only when the surface of the metal is perfectly dry. No painting shall be done in wet or freezing weather unless special precautions are taken.

141. Pins, pin holes, screw threads and other finished surfaces shall be coated with white lead and tallow before being shipped from the shop.

INSPECTION.

Inspection

142. All facilities for inspection of material and workmanship shall be furnished by the contractor to competent inspectors, and the engineer and his inspectors shall be allowed free access to any part of the works in which any portion of the material is made.

143. The contractor shall furnish, without charge, such specimens (prepared) of the several kinds of material to be used as may be required to determine their character.

Testing.

144. Full sized parts of the structure may be tested at the option of the purchaser; but, if tested to destruction, such material shall be paid for at cost, less its scrap value, if it proves satisfactory.

145. If it does not stand the specified tests, it will be considered rejected material, and be solely at the cost of the contractor.

ERECTION.

Tools.

146. The contractor shall furnish at his own expense all necessary tools, staging and material of every description required for the erection of the work, and remove the same when the work is completed.

Risks.

147. The contractor shall assume all risks from storms or accidents, unless caused by the negligence of the owner,

and all damage to adjoining property and to persons until the work is completed and accepted.

The contractor shall comply with all ordinances or regulations appertaining to the work.

The erection must be carried forward with diligence and must be completed promptly.

APPENDIX II.

PROBLEMS IN GRAPHIC STATICS AND THE CALCULATION OF STRESSES.

Introduction.—It is impossible for the student to gain a working knowledge of graphic statics and the calculation of stresses without solving numerous problems. In order to save the time of the student and the instructor the problems must be selected with care, and the data put in working form. The following problems have been given by the author, in connection with a course preliminary to bridge analysis, and are presented here with the hope that they may prove of value to both students and instructors. By slightly changing the quantities and dimensions the data for new problems may be easily obtained.

Instructions.—(1) Plate.—The standard plate is to be 9″ × 10½″, with a 1″ border on the left-hand side, and a ½″ border on the top, bottom, and right-hand side of the plate. The plate inside the border is to be 7½″ × 9½″. (2) Co-ordinates.—Unless stated to the contrary, co-ordinates given in the data will refer to the lower left-hand corner of border as the origin of co-ordinates. In defining the force, $P\frac{1500}{150°}$ (5.0″, 3.0″), the force is 1500 lbs., makes 150° with the X-axis (lies in the second quadrant), and passes through a point 5.0″ to the right, and 3.0″ above the lower left-hand corner of border. (3) Data.—Complete data shall be placed on each problem so that the solution will be self explanatory. (4) Scales.—The scales of forces, and of frames or trusses shall be given as 1″ = ( ) lbs., or ft.; and by a graphic scale as well. (5) Name.—The name of the student is be placed outside the border in the lower right-hand corner. (6) Equations.—All equations shall be given, but details of the solution may be indicated. (7) References.—References are to "The Design of Steel Mill Buildings."

Note.—It should be noted that all the problems have been reduced so that all dimensions are one-half the original dimensions given in the statements of the problems.

PROBLEM 1. RESULTANT OF CONCURRENT FORCES.

(a) Problem.—Given the following concurrent forces:

$$P_1 \frac{1900}{120^\circ}; \qquad P_2 \frac{950}{30^\circ}; \qquad P_3 \frac{1700}{315^\circ}; \qquad P_4 \frac{750}{240^\circ}; \qquad P_5 \frac{2200}{150^\circ}.$$

Forces are given in pounds. Required to find the resultant, R, by means of a force diagram. Check by calculating R by the algebraic method. Give all equations. Also construct check force polygon. Give amount and direction of the resultant. Scale for force polygon, $1'' = 400$ lbs.

(b) Methods.—Start P_1 at point $4''$, $3\frac{1}{2}''$ (x, y, using lower left-hand corner of the border as the origin of co-ordinates) and take the forces in the order, P_1, P_2, P_3, P_4, P_5. Draw check polygon starting at the same point and drawing the forces in the order P_2, P_4, P_5, P_3, P_1.

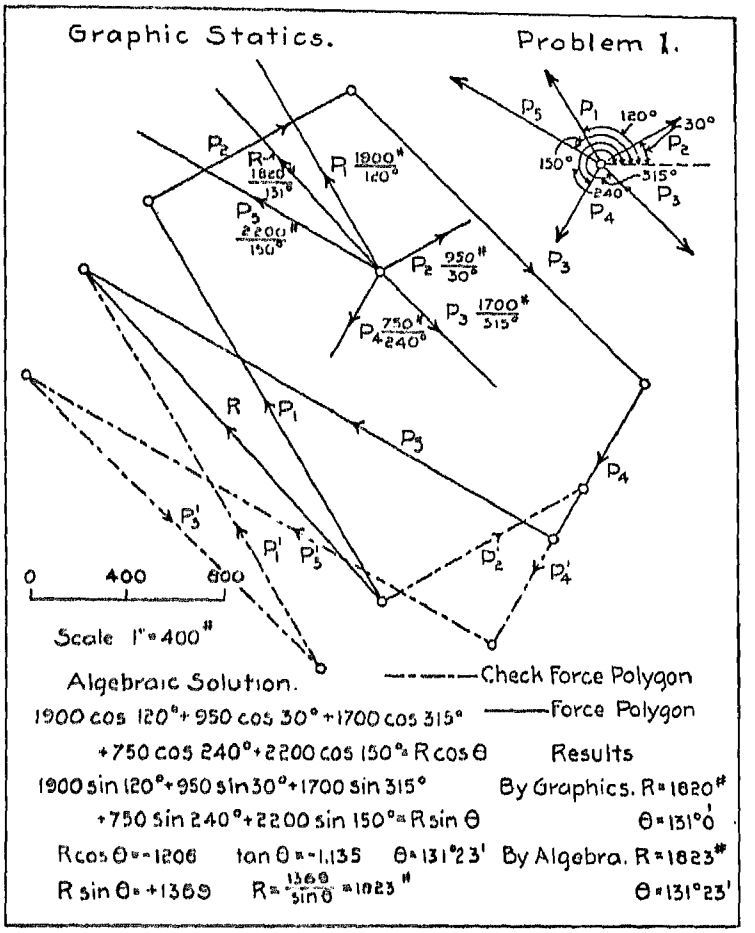
(c) Results.—The resultant is a force R acting through the point of intersection of the given forces, and is parallel to the closing line in the force polygon. It will be seen that it is immaterial in what order the forces are taken in calculating the resultant, R. In the algebraic solution the summation of the horizontal components of the forces, including the resultant, R, are placed equal to zero, and the summation of the vertical components of the forces, including the resultant, R, are placed equal to zero. Solving these equations we have the value of R, and the angle θ, which R makes with the X-axis.

PROBLEM 1a. RESULTANT OF CONCURRENT FORCES.

(a) Problem.—Given the following concurrent forces:

$$P_1 \frac{1800}{120^\circ}; \qquad P_2 \frac{1000}{30^\circ}; \qquad P_3 \frac{1700}{315^\circ}; \qquad P_4 \frac{750}{240^\circ}; \qquad P_5 \frac{2100}{150^\circ}.$$

Forces are given in pounds. Required to find the resultant, R, by means of a force diagram. Check by calculating R by the algebraic method. Give all equations. Also construct check force polygon. Give amount and direction of the resultant. Scale for force polygon, $1'' = 400$ lbs.


Graphic Statics.
Problem 1.
P1 1900# 120°
P2 950# 30°
P3 1700# 315°
P4 750# 240°
P5 2200# 150°
R 1820# 131°
120°
30°
150°
315°
240°
0 400 800
Scale 1" = 400#
Algebraic Solution.
1900 cos 120° + 950 cos 30° + 1700 cos 315°
+750 cos 240° + 2200 cos 150° = R cos θ
1900 sin 120° + 950 sin 30° + 1700 sin 315°
+750 sin 240° + 2200 sin 150° = R sin θ
R cos θ = -1206
R sin θ = +1369
tan θ = -1.135
θ = 131°23'
R = 1369/sin θ = 1823#
Check Force Polygon
Force Polygon
Results
By Graphics. R = 1820#
θ = 131°0'
By Algebra. R = 1823#
θ = 131°23'

PROBLEM 2. RESULTANT OF NON-CONCURRENT FORCES.

(a) **Problem.**—Given the following non-concurrent forces:

$$P_1 \frac{720}{150^\circ}\ (7.0'', 6.3''),\quad P_2 \frac{270}{0^\circ}\ (0.5'', 9.3''),\quad P_3 \frac{350}{205^\circ}\ (7.0'', 6.6''),$$

$$P_4 \frac{210}{15^\circ}\ (0.8'', 8.5'').$$

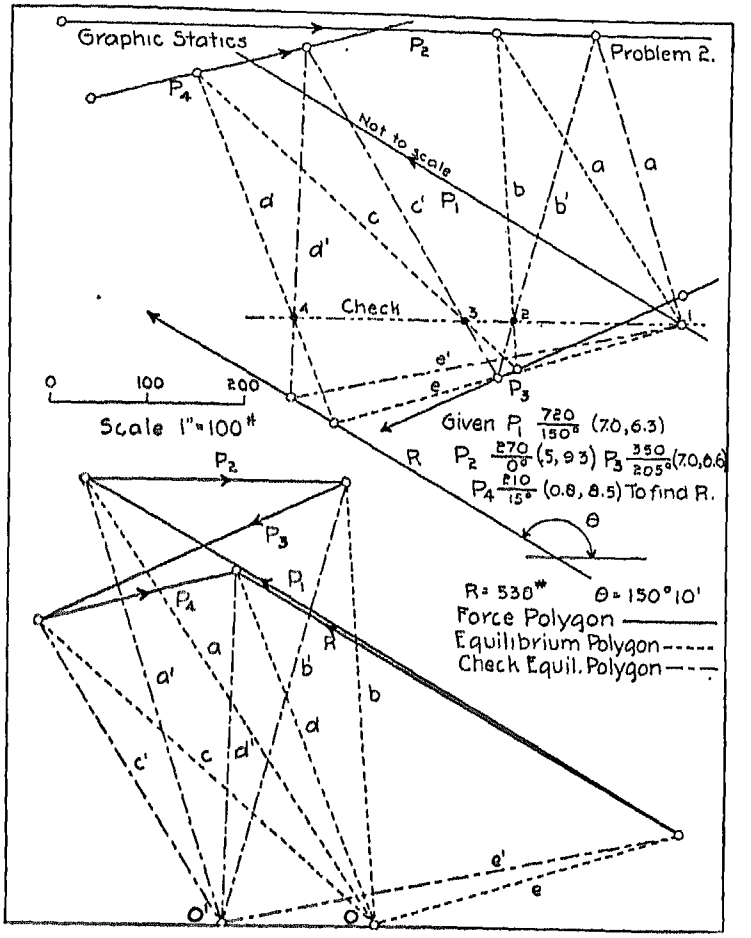
Forces are given in pounds. Find resultant, R, by means of force and equilibrium polygons. Check by calculating R by means of a new force polygon and a new equilibrium polygon. Also check as described below. Scale for force polygon, 1" = 100 lbs.

(b) **Methods.**—Start force polygon at (7.0", 1.0"). Take pole at (3.8", 0.0"). Start equilibrium polygon at (7.0", 6.3"). Take new pole at (2.2", 0.0"), and draw new polygon starting at (7.0", 6.3").

(c) **Results.**—The resultant is a force, R, and acts through the intersection of the strings d, e and d', e', and is parallel to the closing line in force polygon. If corresponding strings in equilibrium polygon are produced to an intersection, the points of intersection, 1, 2, 3, 4, 5, will lie in a straight line which will be parallel to the line O–O' joining the poles of the force polygons. This relation is due to the reciprocal nature of the force and equilibrium polygons, and may be proved as follows: In the force polygon the force P_4 may be resolved into the rays c and d, it may likewise be resolved into the rays c' and d'. In like manner it may be seen that the force O–O' can be resolved into the d and d', or into c and c'. Now if the strings d and d' are drawn through the point 4, and the strings c and c' are drawn, they must intersect in the point 3, and 4–3 must be parallel to O–O'. For the resultant of d and d' is equal to O–O' and must act in a line parallel to O–O'; likewise the resultant of c and c' is equal to O–O' and must act parallel to O–O'; and in order to have equilibrium 3–4 must be parallel to O–O'.

In like manner it may be proved that 1, 2, 3, 4, 5, are in a straight line parallel to O–O'.

From the above it will be seen that to have equilibrium in a system of non-concurrent forces it is necessary that the force polygon and its corresponding equilibrium polygons must close, or that two equilibrium polygons must close.



PROBLEM 2a. RESULTANT OF NON-CONCURRENT FORCES.

(a) Problem.—Given the following non-concurrent forces:

$$P_1 \frac{700}{150^\circ} (7.0'', 6.3''), \quad P_2 \frac{270}{00^\circ} (0.5'', 9.3''), \quad P_3 \frac{350}{205^\circ} (7.0'', 6.6''),$$

$$P_4 \frac{300}{15^\circ} (0.8'', 8.5'').$$

The rest of the statement is the same as for Problem 2.

PROBLEM 3. TEST OF EQUILIBRIUM OF FORCES.

(a) Problem.—Test the following forces for equilibrium by means of force and equilibrium polygons:

$$P_1 \frac{7.8}{90^\circ}(4.75'', 0.0'');\quad P_2 \frac{10.8}{45^\circ}(4.75'', 7.7'');\quad P_3 \frac{12.2}{0^\circ}(0.0'', 7.0'');$$

$$P_4 \frac{13.2}{300^\circ}(4.75'', 6.4'');\quad P_5 \frac{19.35}{210^\circ}(2.75'', 6.4'');\quad P_6 \frac{11.3}{150^\circ}(2.75'', 7.7'').$$

The forces are given in tons. Check by using a second pole and equilibrium polygon; also draw a line through the intersection of the corresponding rays, and check as in Problem 2. Give amount and direction of the equilibrant. Scale of forces, $1'' = 5$ tons.

(b) Methods.—Start force polygon at (0.8", 1.5"). Take first pole at (3.2", 2.7"). Start equilibrium polygon at (4.75", 9.1"). Take second pole at (4.0", 3.2"). Start second equilibrium polygon at (4.75", 8.2").

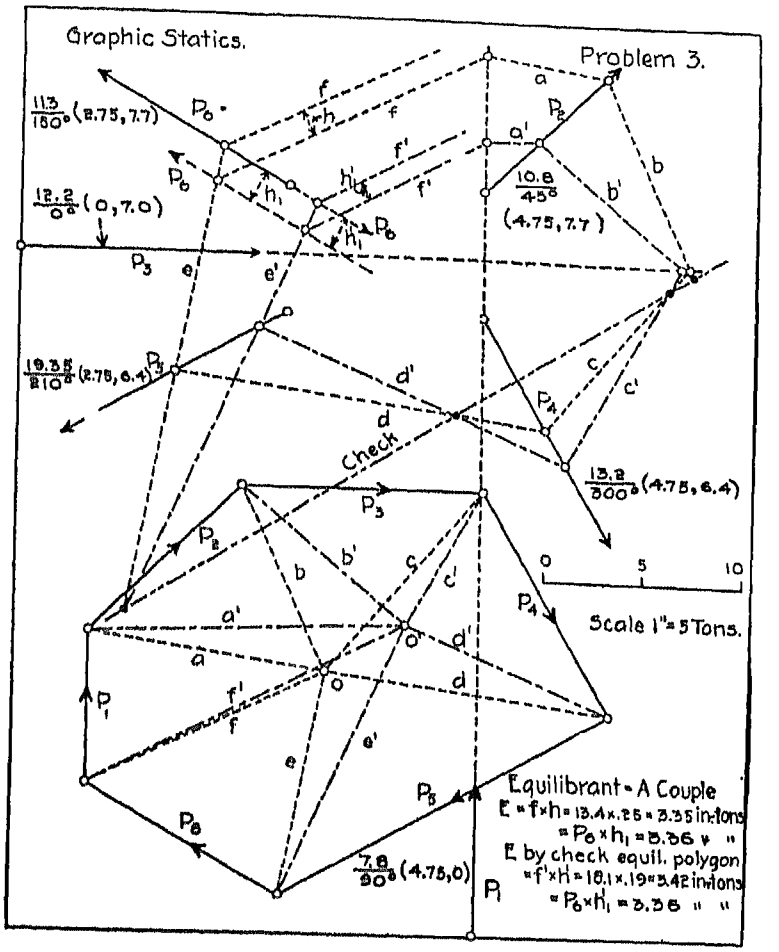
(c) Results.—If the system of forces was in equilibrium the equilibrium polygons would close, and the first and last strings f and f, and f' and f' would coincide, respectively. The equilibrant will be equal to a couple with a moment represented by the rays f or f' multiplied by the distance h or h'. In general in any system of non-current forces if the force polygon closes the equilibrant of the system is a couple. If the system is in equilibrium the arm of the couple is zero. It is evident that in order that any system of non-concurrent forces be in equilibrium it is necessary that both the force polygon and an equilibrium polygon must close. The check line must be parallel to the line O–O' joining the poles, and also pass through the intersections of corresponding rays as in Problem 2.

PROBLEM 3a. TEST OF EQUILIBRIUM OF FORCES.

(a) Problem.—Test the following forces for equilibrium by means of force and equilibrium polygons:

$$P_1 \frac{7.8}{90^\circ}(4.75'', 0.0'');\quad P_2 \frac{10.8}{45^\circ}(4.75'', 7.7'');\quad P_3 \frac{13.93}{0^\circ}(0.0'', 7.0'');$$

$$P_4 \frac{13.2}{300^\circ}(4.75'', 6.4'');\quad P_5 \frac{20.35}{210^\circ}(2.75'', 6.4'');\quad P_6 \frac{12.3}{150^\circ}(2.75'', 7.7'').$$



The forces are given in tons. Check by using a second pole and equilibrium polygon; also draw a line through the intersection of the corresponding rays, and check as in Problem 2. Give amount and direction of the equilibrant. Scale of forces, 1″ = 5 tons.

PROBLEM 4. RESOLUTION OF FORCES.

(a) Problem.—Given the following forces:

$$P_1 \frac{13.2}{270^\circ} (1.0'', —); \quad P_2 \frac{5.0}{270^\circ} (2.8'', —); \quad P_3 \frac{3.2}{270^\circ} (6.8'', —).$$

Forces are given in tons. (1) Find the resultant, R, by means of force and equilibrium polygons. (2) Resolve R into two parallel forces

$$P' \frac{?}{270^\circ} (3.2'', —) \text{ and } P''' \frac{?}{90^\circ} (5.4'', —).$$ (3) Find the moment, M, of R about a point Z at (4.0", —). Check by the algebraic method giving all equations. Scale of forces, 1" = 5 tons.

(b) Methods.—Start force polygon at (5.7", 0.5"), and take pole at (1.7", 3.0"). In the algebraic method take moments about the left border in finding the point of application of R, and take moments in the line of action of P''' in resolving R into P'' and P'''.

(c) Results.—(1) The position of R is at the intersection of strings a and d. (2) Prolong string a until it intersects P', take the intersection of string d with P'''; then string c is the closing line of the polygon. (3) The moment of R is found graphically by multiplying the intercept, y, by the pole distance, H. (4) To find position of R algebraically take moment of P_1, P_2, and P_3, about the left border, and divide by R, which is the sum of P_1, P_2, P_3. (5) The moment of R about Z is equal to $R \times (4.0 - 2.28)''$, = 36.8 in.-tons.

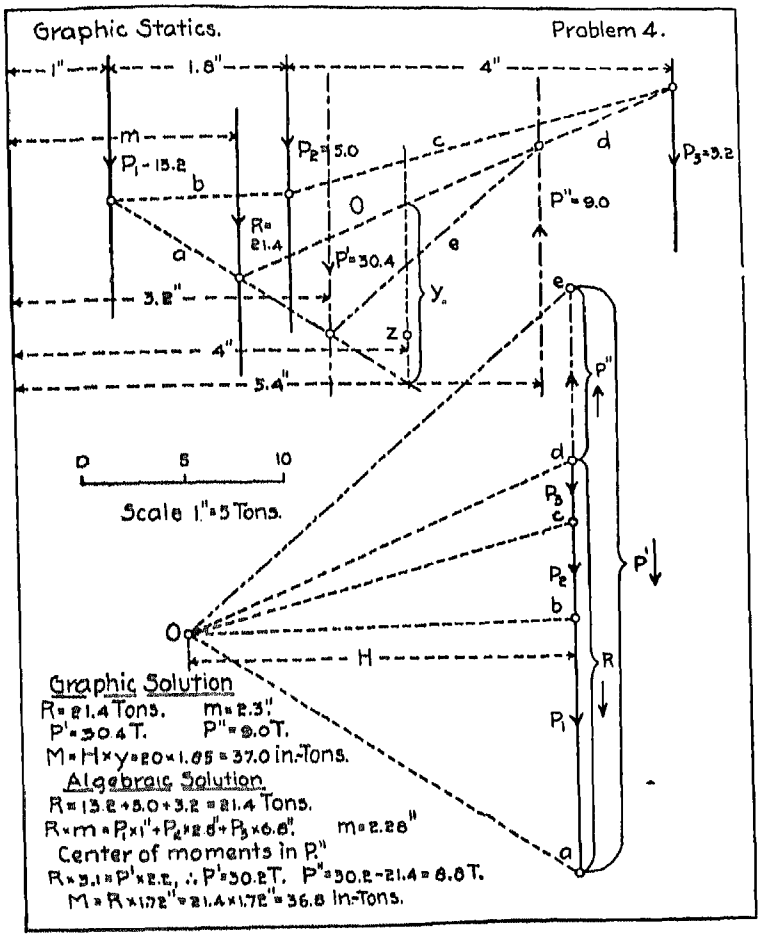
PROBLEM 4a. RESOLUTION OF FORCES.

(a) Problem.—Given the following forces:

$$P_1 \frac{13.2}{270^\circ} (1.0'', —); \quad P_2 \frac{5.0}{270^\circ} (2.8'', —); \quad P_3 \frac{4.0}{270^\circ} (7.0'', —).$$

Forces are given in tons. (1) Find the resultant, R, by means of force and equilibrium polygons. (2) Resolve R into two parallel forces

$$P' \frac{?}{270^\circ} (3.2'', —) \text{ and } P''' \frac{?}{90^\circ} (5.4'', —).$$ (3) Find the moment, M, of R about a point Z at (4.5", —). Check by the algebraic method giving all equations. Scale of forces, 1" = 5 tons.


Graphic Statics.
Problem 4.
1"
1.8"
4"
m
P1 = 13.2
P2 = 5.0
P3 = 3.2
b
c
d
O
P" = 9.0
R = 21.4
a
P' = 30.4
e
3.2"
y
z
4"
5.4"
e
P"
0 5 10
Scale 1" = 5 Tons.
d
P3
c
P'
P2
b
O
H
R
Graphic Solution
R = 21.4 Tons. m = 2.3"
P' = 30.4 T. P" = 9.0 T.
M = H × y = 20 × 1.85 = 37.0 in.-Tons.
Algebraic Solution
R = 13.2 + 5.0 + 3.2 = 21.4 Tons.
R × m = P1 × 1" + P2 × 2.8" + P3 × 6.8". m = 2.28"
Center of moments in P."
R × 3.1 = P' × 2.2, ∴ P' = 30.2 T. P" = 30.2 - 21.4 = 8.8 T.
M = R × 1.72" = 21.4 × 1.72" = 36.8 in.-Tons.
P1
a

PROBLEM 5. CENTER OF GRAVITY OF AN AREA.

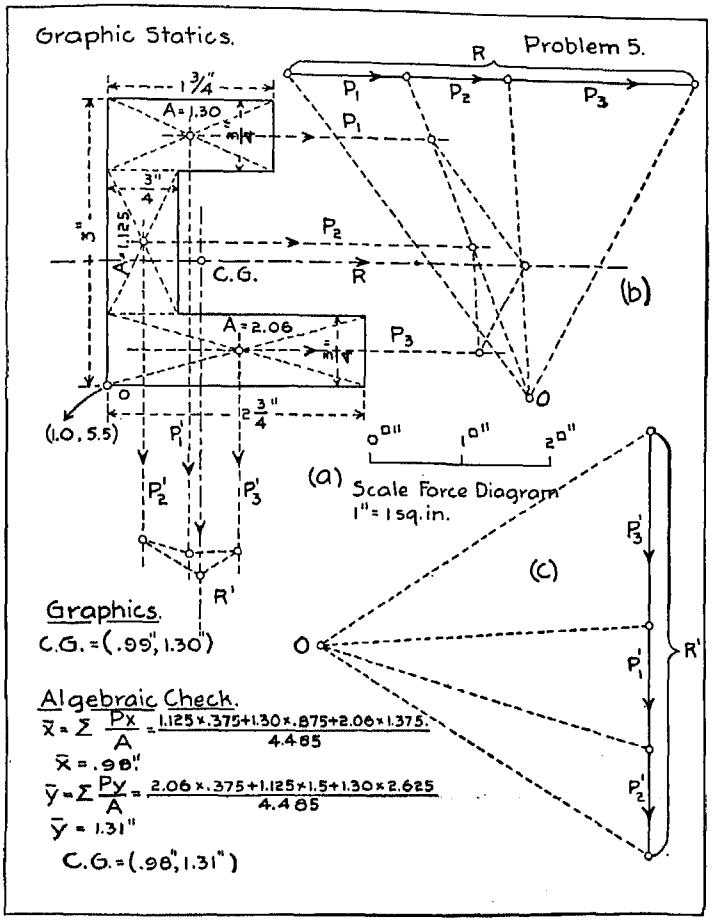
(a) Problem.—Find the center of gravity of the given figure about the X- and Y-axes by graphics. Give the co-ordinates of the C. G. referred to O as the origin. Show all force and equilibrium polygons. Check by the algebraic method stating all equations. Scale of figure, $1'' = 1''$. Scale of forces, $1'' = 1$ sq. in.

(b) Methods.—Start force polygon (b) at point (2.9″, 8.8″) and take pole at (5.6″, 5.4″). Start force polygon (c) at (6.9″, 0.6″), and take pole at (3.25″, 2.8″). In the algebraic check take moments about the left-hand edge and the lower edge of the figure.

(c) Results.—The center of gravity of the figure will come at the intersection of the resultants R and R', which is at the center of area. The areas P_1, P_2, and P_3, may be taken as acting at any angle, but maximum accuracy is attained when the forces are assumed as acting at right angles. If the figure has an axis of symmetry (an axis such that every point on one side of the axis has a corresponding point on the other side at the same distance from the axis) but one force and equilibrium polygon is required.

PROBLEM 5a. CENTER OF GRAVITY OF AN AREA.

(a) Problem.—Find the center of gravity of a $6'' \times 4'' \times 1''$ angle with the long leg vertical and short leg to the right about the X- and Y-axes by graphics. Give the co-ordinates of the C. G. referred to O as the origin. Show all force and equilibrium polygons. Check by the algebraic method stating all equations. Scale of figure, $1'' = 2''$. Scale of forces, $1'' = 2$ sq. in.


Graphic Statics.
Problem 5.
R
P1
P2
P3
1 3/4"
A=1.30
3"
3/4"
A=1.125
C.G.
(b)
A=2.06
2 3/4"
(1.0, 5.5)
0
1"
2"
(a) Scale Force Diagram
1" = 1 sq. in.
(c)
R'
Graphics.
C.G. = (.99", 1.30")
Algebraic Check.
$\bar{x} = \Sigma \frac{Px}{A} = \frac{1.125\times.375+1.30\times.875+2.06\times1.375}{4.485}$
$\bar{x} = .98"$
$\bar{y} = \Sigma \frac{Py}{A} = \frac{2.06\times.375+1.125\times1.5+1.30\times2.625}{4.485}$
$\bar{y} = 1.31"$
C.G. = (.98", 1.31")

PROBLEM 6. MOMENT OF INERTIA OF AN AREA.

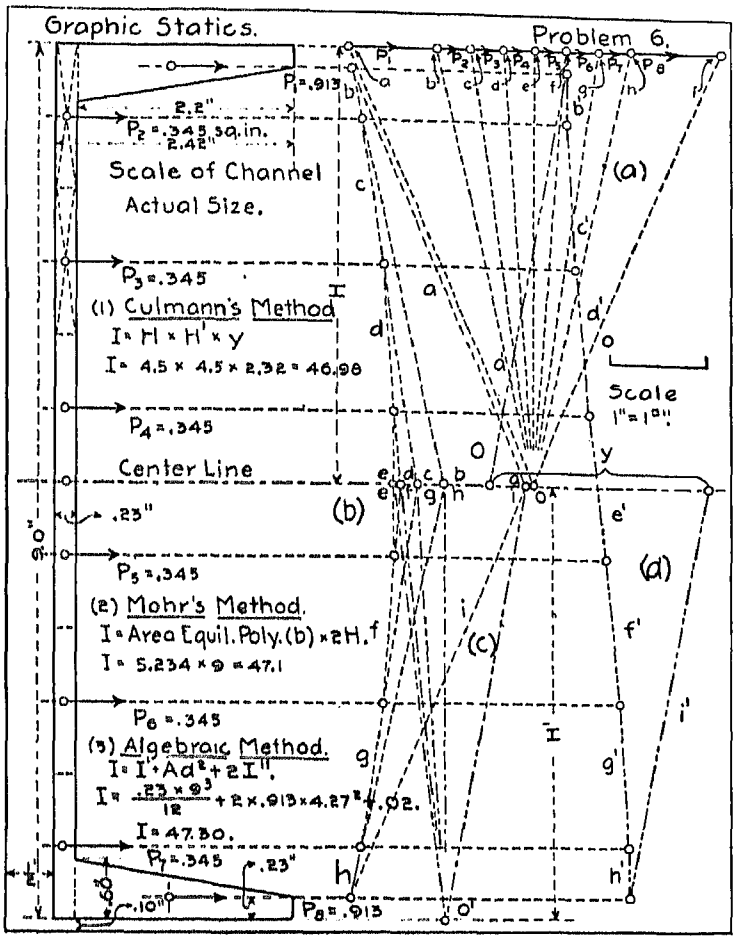
(a) Problem. Calculate the moment of ineltia, I, of a standard 9″ [@ 13.25 lbs., about an axis through its center at right angles to the web: (1) By Culmann's approximate method; (2) by Mohr's approximate method; (3) by the algebraic method. Omit the fillets. Scale of channel, 1″ = 1″. Scale of forces, 1″ = 1 sq. in.

(b) Methods.—Divide the channel into convenient sections and consider the areas as forces acting through their centers of gravity. (1) Culmann's method (Fig. 22). Start force polygon (a) at (3.5″, 9.1″), and take pole at (5.45″, 4.6″). Draw equilibrium polygon (b). Now with intercepts *a–b, b–c, c–d, d–e, e–f, f–g, g–h, h–i,* as forces, and a new pole at (4.5″, 0.1″) construct equilibrium polygon (d). The moment of inertia is (approximately) $I = H \times H' \times y$. (2) Mohr's method (Fig. 23). Calculate the area of the equilibrium polygon (b) by means of the planimeter or by dividing it into triangles and (approximately) I = area equilibrium polygon (b) $\times 2H$. If the area is divided into an infinite number of sections, or if the true curve of equilibrium be drawn through the points determined, this method gives the true value of I. (3) Algebraic method. The moment of inertia about the center line is $I = I' + A\,d^2 + 2\,I''$ where I' = moment of inertia of the main rectangle; A = area of the two flanges; d = distance of the center of gravity of the flanges from the center line; and I'' = moment of inertia of each flange about an axis through its center of gravity parallel to the center line.

(c) Results.—The algebraic method gives the true value of I; Mohr's method gives a value more nearly correct than Culmann's method, as would have been expected. The values of I given in the various hand-books are calculated by the algebraic method.

PROBLEM 6a. MOMENT OF INERTIA OF AN AREA.

(a) Problem.—Calculate the moment of inertia, I, of a standard 9″ [@ 15 lbs., about an axis through its center at right angles to the web: (1) By Culmann's approximate method; (2) by Mohr's approximate method; (3) by the algebraic method. Omit the fillets. Scale of channel, 1″ = 1″. Scale of forces, 1″ = 1 sq. in.


Graphic Statics.
Problem 6.
P1=.913
2.2"
P2=.345 sq.in.
2.42"
Scale of Channel
Actual Size.
P3=.345
(1) Culmann's Method
I = H × H' × y
I = 4.5 × 4.5 × 2.32 = 46.98
P4=.345
Center Line
.23"
P5=.345
(2) Mohr's Method.
I = Area Equil. Poly. (b) × 2H. f
I = 5.234 × 9 = 47.1
P6=.345
(3) Algebraic Method.
I = I' + Ad² + 2I''.
I = .23 × 9³/12 + 2 × .913 × 4.27² + .02.
I = 47.80.
P7=.345
.23"
.10"
P8=.913
Scale
1"=1"
(a)
(b)
(c)
(a)

PROBLEM 7. CONSTRUCTION OF AN INERTIA ELLIPSE AND AN INERTIA CIRCLE.

(a) Problem.—Given the following data for an angle $7'' \times 3\frac{1}{2}'' \times 1''$; $A = 9.50$ sq. in., $I_1 = 7.53''^4$, $I_2 = 45.37''^4$; $r_1 = 0.89''$; $r_2 = 2.19''$; $r_3 = 0.74''$; tan $\alpha = 0.241$; C. G. $(2.71'', 0.96'')$, see Cambria, pp. 176, 177. (1) Construct the inertia ellipse. (2) Construct the inertia circle. Omit the fillets. Scale of the angle, $1'' = 1''$.

(b) Methods. (1) *Inertia Ellipse.*—Construct angle α, tan $\alpha = 0.241$; and draw axes 3–3 and 4–4, which are the principal axes of the inertia ellipse. Calculate r_4 from the relation $I_1 + I_2 = I_3 + I_4$, from which $r_1^2 + r_2^2 = r_3^2 + r_4^2$, and $r_4 = 2.25''$. Construct the enclosing rectangle of the ellipse on the axes 3–3 and 4–4, and inscribe an ellipse in this rectangle; this ellipse is the central inertia ellipse.

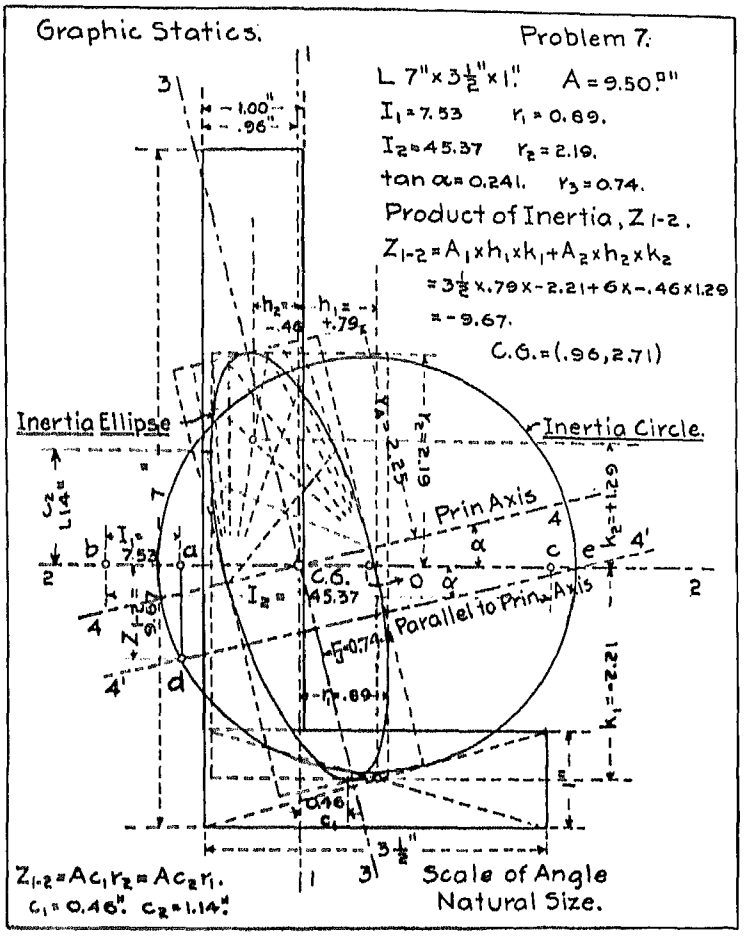
Calculate $Z_{1\text{-}2}$ from the relation $Z_{1\text{-}2} = A_1 \times h_1 \times k_1 + A_2 \times h_2 \times k_2$. Also calculate c_1 and c_2 from the relation $Z_{1\text{-}2} = Ac_1r_2 = Ac_2r_1$. Compare the calculated values of c_1 and c_2 with the scaled values on the ellipse. Note that c_1 and c_2 are zero for the principal axes.

(2) *Inertia Circle.*—Calculate the product of inertia, $Z_{1\text{-}2} = -9.67$. From any given point, a, lay off $I_1 = 7.53$ to the left extending to b, lay off $I_2 = 45.37$ to the right from b, and extending to c. At a erect a perpendicular $a\text{–}d = Z_{1\text{-}2} = -9.67$. Then with center O, midway between a and c, and with a radius $O\text{–}d$ describe a circle, which will be the inertia circle. A line drawn through d and c will be parallel to the principal axis 4–4, and the diameter of the inertia circle will be the maximum value of $I_2 - I_1$.

(c) Results.—(1) The inertia ellipse drawn is the central ellipse of inertia, and is the smallest ellipse that can be drawn. The radii of gyration about any axis can be found directly from the inertia ellipse. (2) The moments of inertia about any axis can be found directly from the circle of inertia.

PROBLEM 7a. CONSTRUCTION OF AN INERTIA ELLIPSE AND AN INERTIA CIRCLE.

(a) Problem.—Given the data for an angle $7'' \times 3\frac{1}{2}'' \times \frac{7}{8}''$; see Cambria, pp. 176, 177. (1) Construct the inertia ellipse. (2) Construct the inertia circle. Omit the fillets. Scale of the angle, $1'' = 1''$.



References: "The Determination of Unit Stresses in the General Case of Flexure" by Professor L. J. Johnson in Assoc. Eng. Soc., Vol. XXVIII; Appendix D. Maurer's "Technical Mechanics"; Muller-Breslau's "Graphische Statik der Baukonstruktionen," Band I.

PROBLEM 8. STRESSES IN A ROOF TRUSS BY GRAPHIC AND ALGEBRAIC RESOLUTION.

(a) Problem.—Given a Fink truss, span 40′–0″, pitch 30°; trusses spaced 12′–0″; load 40 lbs. per sq. ft. of horizontal projection. Calculate the reactions by means of force and equilibrium polygons. Calculate the stresses by graphic resolution, and check by algebraic resolution. Scale of truss, 1″ = 8′–0″. Scale of loads, 1″ = 6000 lbs.

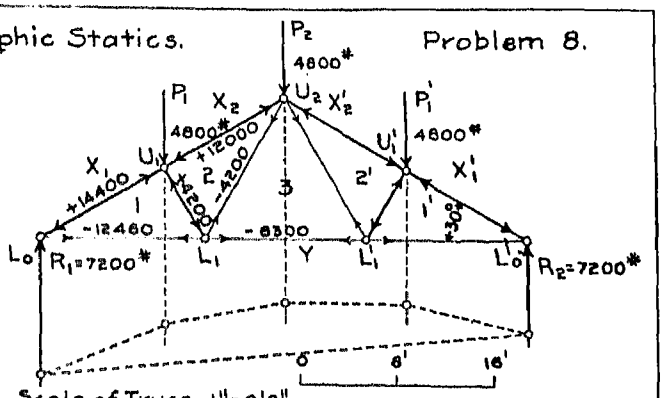
(b) Methods.—Start force polygon at (6.25″, 5.6″). Lay off the loads in the order P_1, P_2, P'_1, from the top downward. Construct a force polygon, and draw an equilibrium polygon as in Fig. 14, and calculate the reactions R_1 and R_2 by means of the closing line as in Fig. 15. Construct stress diagram beginning at L_0 and analyzing the joints in the order, L_0, U_1, L_1, U_2, etc., checking at L'_0. Arrows acting toward joints in the truss and toward the ends of the lines in the stress diagram indicate compression, while arrows acting away from the joints and ends of lines respectively, indicate tension. Use one arrow in the stress diagram the first time a force is used, and two arrows the second time. In algebraic resolution the sum of the horizontal components at any joint are placed equal to zero, and the sum of the vertical components are placed equal to zero, and the solution of these two sets of equations gives the required stresses.

(c) Results.—The top chord is in compression, while the bottom chord is in tension. In the Fink truss it will be seen that the long web members are in tension, while the short web members are in compression. This makes the truss a very economical one.

PROBLEM 8a. STRESSES IN A ROOF TRUSS BY GRAPHIC AND ALGEBRAIC RESOLUTION.

(a) Problem.—Given a Fink truss, span 40′–0″, pitch ⅓; trusses spaced 14′–0″; load 40 lbs. per sq. ft. of horizontal projection. Calculate the reactions by means of force and equilibrium polygons. Calculate the stresses by graphic resolution, and check by algebraic resolution. Scale of truss, 1″ = 8′–0″. Scale of loads, 1″ = 5000 lbs.

Graphic Statics. Problem 8.

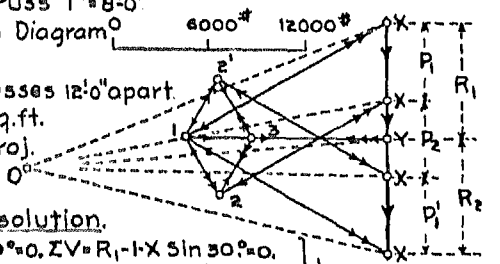


Scale of Truss 1"=8'-0"

Scale of Stress Diagram 1"=6000#

Span 40'-0". Trusses 12'-0" apart.

Load=40# per sq.ft. horizontal proj.



Algebraic Resolution.

$\Sigma H = 1\text{-}Y + 1\text{-}X \cos 30° = 0$. $\Sigma V = R_1 - 1\text{-}X \sin 30° = 0$.
$\therefore 1\text{-}Y = -12470^{\#}$ $\therefore 1\text{-}X = +14400^{\#}$ } L_0

$\Sigma H = 1\text{-}X \cos 30° - 2\text{-}X \cos 30° - 1\text{-}2 \cos 60° = 0$.
$\Sigma V = 4800 + 2\text{-}X \sin 30° - 1\text{-}X \sin 30° - 1\text{-}2 \sin 60° = 0$.
$\therefore 1\text{-}2 = 4160^{\#}$ $\therefore 2\text{-}X = 12000^{\#}$ } U_1

$\Sigma H = -1\text{-}Y + 3\text{-}Y + 1\text{-}2 \cos 60° + 2\text{-}3 \cos 60° = 0$.
$\Sigma V = -1\text{-}2 \sin 60° + 2\text{-}3 \sin 60° = 0$. $\therefore 2\text{-}3 = -4160^{\#}$. $\therefore 3\text{-}Y = -8310^{\#}$ } L_1

Summary.

Algebraic. $1\text{-}X = +14400^{\#}$, $2\text{-}X = +12000^{\#}$, $1\text{-}Y = -12470^{\#}$, $3\text{-}Y = -8310^{\#}$, $1\text{-}2 = 2\text{-}3 = 4160^{\#}$.

Graphic. $1\text{-}X = +14400^{\#}$, $2\text{-}X = +12000^{\#}$, $1\text{-}Y = -12460^{\#}$, $3\text{-}Y = -8300^{\#}$, $1\text{-}2 = 2\text{-}3 = 4200^{\#}$.

PROBLEM 9. DEAD LOAD STRESSES IN A TRIANGULAR TRUSS BY GRAPHIC RESOLUTION.

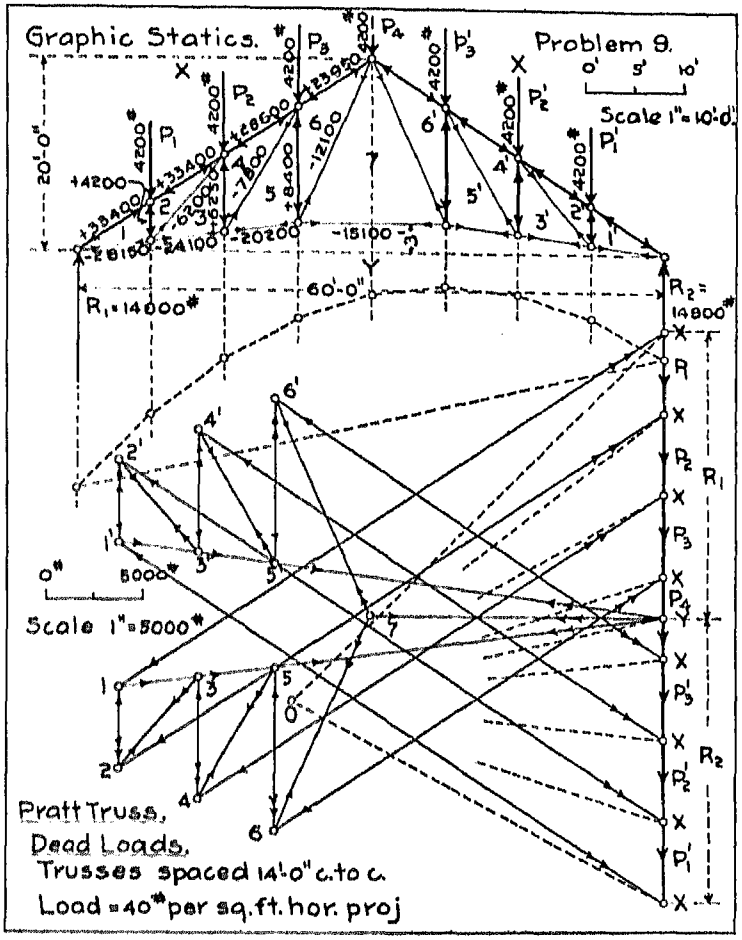
(a) **Problem.**—Given a triangular truss, span 60′–0″; pitch ⅓; camber of the bottom chord 3′–0″; trusses spaced 14′–0″; load 40 lbs. per sq. ft. of horizontal projection. Calculate the reactions by means of force and equilibrium polygons. Calculate the stresses by graphic resolution. Scale of truss, 1″ = 10′–0″. Scale of loads, 1″ = 5000 lbs.

(b) **Methods.**—Start the truss at (0.75″, 7.0″). Start the stress diagram at (6.75″, 6.25″). Calculate the stresses beginning at R_1, as described in Fig. 27, using care to analyze each joint before proceeding to the next. Check at R_2.

(c) **Results.**—The upper chord is in compression while the lower chord is in tension. The vertical web members are in compression while the inclined web members are in tension for dead loads. As a check the points 2, 4, 6, should be in a straight line. The partial loads coming on the reactions (not shown) are not considered as they have no effect on the stresses in the truss. This truss is a triangular Pratt and is quite economical, but is somewhat more expensive in material and labor than the Fink truss. This type of truss is much used for combination trusses, in which the tension members are made of iron or steel, while the compression members are made of timber. The dead joint load will be equal to the horizontal projection of the area supported by a panel point, multiplied by the dead load per square foot, is equal to $14 \times 7\frac{1}{2} \times 40 = 4200$ lbs.

PROBLEM 9a. DEAD LOAD STRESSES IN A TRIANGULAR TRUSS BY GRAPHIC RESOLUTION.

(a) **Problem.**—Given a triangular truss, span 60′–0″; pitch ⅓; camber of the bottom chord 2′–0″; trusses spaced 16′–0″; load 40 lbs. per sq. ft. of horizontal projection. Calculate the reactions by means of force and equilibrium polygons. Calculate the stresses by graphic resolution. Scale of truss, 1″ = 10′–0″. Scale of loads, 1″ = 5000 lbs.


Graphic Statics.
Problem 9.
Scale 1"=10'-0"
Scale 1"=5000#
Pratt Truss,
Dead Loads.
Trusses spaced 14'-0" c. to c.
Load = 40# per sq. ft. hor. proj
60'-0"
20'-0"
R1 = 14000#
R2 = 14800#

PROBLEM 10. WIND LOAD STRESSES IN A TRIANGULAR TRUSS—NO ROLLERS—BY GRAPHIC RESOLUTION.

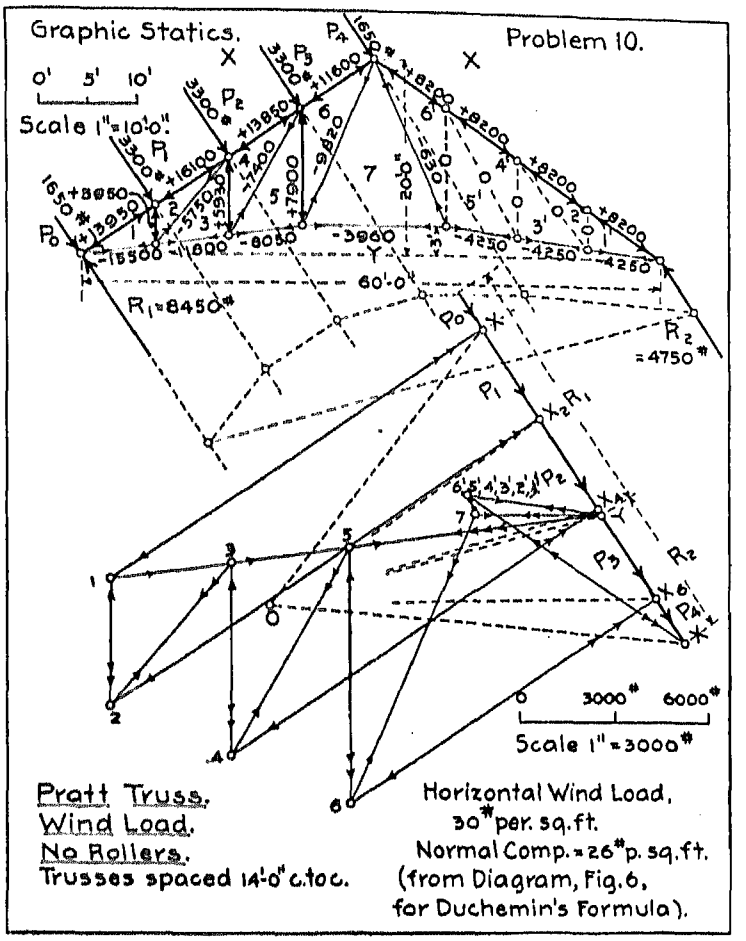
(a) Problem.—Given a triangular truss, span 60′–0″, pitch ⅓, camber of lower chord 3′–0″, trusses spaced 14′–0″, wind load normal component of a horizontal wind load of 30 lbs. per sq. ft., no rollers. Calculate the reactions by means of force and equilibrium polygons. Calculate the stresses by graphic resolution. Take P as 3300 lbs. Scale of truss, 1″ = 10′–0″. Scale of loads, 1″ = 3000 lbs.

(b) Methods.—Start stress diagram at (4.65″, 6.6″). The reactions will be parallel to each other and to the resultant of the external loads. The equilibrium polygon may be started at any convenient point in one reaction, closing up on the other one. The closing line of the equilibrium polygon will always have its end in the reactions. The calculation of stresses is begun at R_1, and is checked up at R_2.

(c) Results.—The stresses are of the same kind in the chords as for dead loads as given in Problem 9, while the webs on the leeward side are not stressed. The load P_0 has no effect on the stresses in the truss. Calculate the vertical component of the wind load by means of Duchemin's formula (5), as plotted in Fig. 6 (page 15). The normal wind joint load will be equal to $14 \times 9 \times 26 = 3276$, which is taken as 3300 lbs. For a discussion on the different conditions of the ends of trusses, see Chapter VII.

PROBLEM 10a. WIND LOAD STRESSES IN A TRIANGULAR TRUSS—NO ROLLERS—BY GRAPHIC RESOLUTION.

(a) Problem.—Given a triangular truss, span 60′–0″, pitch ⅓, camber of lower chord 2′–0″, trusses spaced 16′–0″, wind load normal component of a horizontal wind load of 30 lbs. per sq. ft., no rollers. Calculate the reactions by means of force and equilibrium polygons. Calculate the stresses by graphic resolution. Take P as 3800 lbs. Scale of truss, 1″ = 10′–0″. Scale of loads, 1″ = 3000 lbs.


Graphic Statics.
Problem 10.
0' 5' 10'
Scale 1"=10'0".
60'-0"
R1=8450#
R2 =4750#
Scale 1"=3000#
3000#
6000#
Pratt Truss.
Wind Load.
No Rollers.
Trusses spaced 14'-0" c.to c.
Horizontal Wind Load,
30# per. sq.ft.
Normal Comp.=26#p. sq.ft.
(from Diagram, Fig.6,
for Duchemin's Formula).

PROBLEM 11. DEAD LOAD STRESSES IN A FINK TRUSS BY GRAPHIC RESOLUTION.

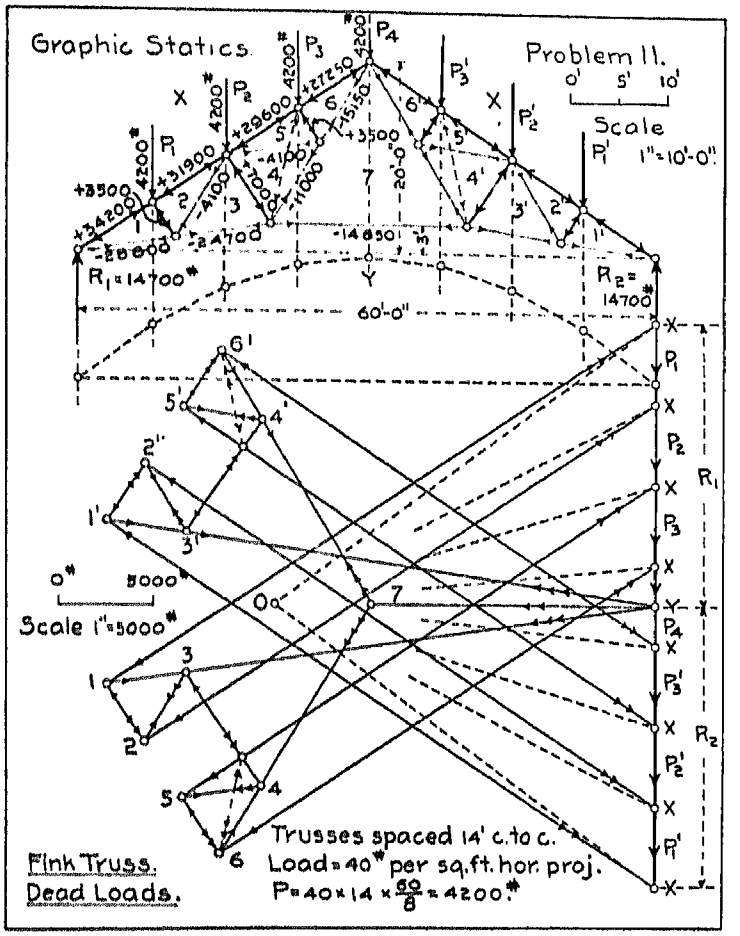
(a) Problem.—Given a Fink truss, span 60′–0″, pitch ⅓, camber of lower truss 3′–0″, trusses spaced 14′–0″, load 40 lbs. per sq. ft. of horizontal projection. Calculate the reactions by means of force and equilibrium polygons. Calculate the stresses by graphic resolution. Scale of truss, 1″ = 10′–0″. Scale of loads, 1″ = 5000 lbs.

(b) Methods.—Start the truss at (0.75″, 7.0″). Start the stress diagram at (6.75″, 6.25″). Calculate the stresses as described in Figs. 30 and 34, replacing the members 4–5 and 5–6, temporarily by the dotted member shown. The stress diagram is then carried through to the point 7, and then the stresses in members 5–6 and 4–5 are easily obtained. Carry the stress diagram through and check at R_2.

(c) Results.—The upper chord is in compression and the lower chord is in tension, the stresses being practically the same as in the triangular truss in Problem 9. In the webs it will be seen that the long members are in tension, while the short members are in compression. The loads coming on the reaction are not considered, as they have no effect on the stresses in the truss, as can be seen by comparing with the truss in Fig. 30. As a check the points 1, 2, 5, 6, in the stress diagram should be in a straight line. The dead joint load will be equal to the horizontal projection of the area supported by a panel point, multiplied by the dead load per square foot, is equal to $14 \times 7\frac{1}{2} \times 40 = 4200$ lbs.

PROBLEM 11a. DEAD LOAD STRESSES IN A FINK TRUSS BY GRAPHIC RESOLUTION.

(a) Problem.—Given a Fink truss, span 60′–0″, pitch ⅓, camber of lower truss 2′–0″, trusses spaced 16′–0″, load 40 lbs. per sq. ft. of horizontal projection. Calculate the reactions by means of force and equilibrium polygons. Calculate the stresses by graphic resolution. Scale of truss, 1″ = 10′–0″. Scale of loads, 1″ = 5000 lbs.


Graphic Statics.
Problem 11.
Scale 1"=10'-0"
Scale 1"=5000#
Fink Truss.
Dead Loads.
Trusses spaced 14' c. to c.
Load = 40# per sq. ft. hor. proj.
P = 40 × 14 × 60/8 = 4200.#
R1 = 14700#
R2 = 14700#
60'-0"

PROBLEM 12. WIND LOAD STRESSES IN A FINK TRUSS—ROLLERS LEEWARD—BY GRAPHIC RESOLUTION.

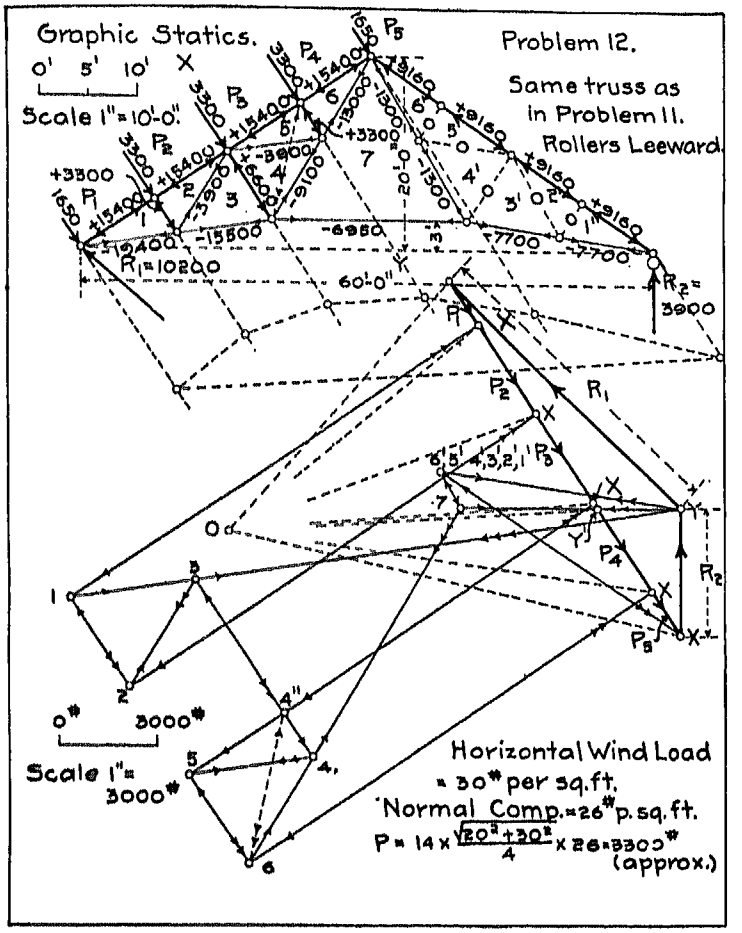
(a) Problem.—Given the same truss as in Problem 11. Wind load to be the normal component of a horizontal wind load of 30 lbs. per sq. ft. The truss is assumed to have frictionless rollers under the leeward side. Calculate the reactions by means of force and equilibrium polygons. Calculate the stresses by graphic resolution. Scale of truss, 1″ = 10′-0″. Scale of loads, 1″ = 3000 lbs.

(b) Methods.—Start stress diagram at (4.75″, 6.55″). The reaction R_2 will be vertical, while the direction of R_1 will be unknown. Use the method of calculating the reactions described on page 51, and in Fig. 33; noting that the vertical components of the reactions are independent of the conditions of the ends of the truss. In calculating the stresses the ambiguity of stresses at point 3–4–7–4–*y* is removed by substituting the dotted member shown, for members 4–5 and 5–6. The calculation of the stresses is begun at R_1, and is checked up at R_2.

(c) Results.—The load P_1 has no effect on the stresses in the truss. The stresses in the members are of the same kind as for dead loads as given in Problem 11, except that there are no stresses in the web members on the leeward side.

PROBLEM 12a. WIND LOAD STRESSES IN A FINK TRUSS—ROLLERS LEEWARD—BY GRAPHIC RESOLUTION.

(a) Problem.—Given the same truss as in Problem 11a. Wind load to be the normal component of a horizontal wind load of 30 lbs. per sq. ft. The truss is assumed to have frictionless rollers under the leeward side. Calculate the reactions by means of force and equilibrium polygons. Calculate the stresses by graphic resolution. Scale of truss, 1″ = 10′-0″. Scale of loads, 1″ = 3000 lbs.


Graphic Statics.
Problem 12.
Same truss as in Problem 11.
Rollers Leeward.
0' 5' 10'
Scale 1"=10'-0".
60'-0"
R1=10200
R2=3000
Scale 1"=3000#
Horizontal Wind Load
= 30# per sq.ft.
Normal Comp.=26# p. sq. ft.
P = 14 x √(20²+30²)/4 x 26 = 3300# (approx.)

PROBLEM 13. WIND LOAD STRESSES IN A FINK TRUSS—ROLLERS ON THE WINDWARD SIDE—BY GRAPHIC RESOLUTION.

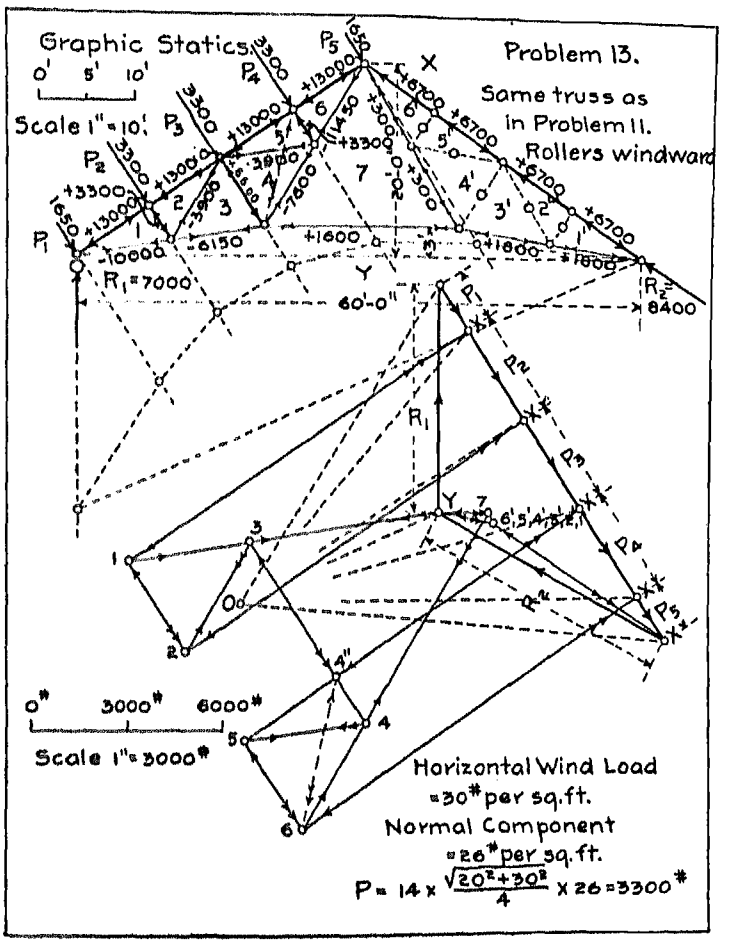
(a) Problem.—Given the same truss and wind load as in Problem 12, and with rollers under the windward side. Calculate the reactions by means of force and equilibrium polygons. Calculate the stresses by graphic resolution. Scale of truss, 1″ = 10′-0″. Scale of loads, 1″ = 3000 lbs.

(b) Methods.—Start stress diagram at (4.6″, 6.7″). Reaction R_1 will be vertical, while the direction of R_2 will be unknown, the only known point in its line of action being at the right end of the truss. Use the method for finding the reactions described on page 52 and in Fig. 34. In this solution the equilibrium polygon is started at the right reaction, the only known point in R_2, and the polygon is drawn. The intersection of R_1 in the force polygon, and a line through O, parallel to the closing line is at Y, and R_2 is then determined in magnitude and direction. The stress diagram is carried through and checked at R_2.

(c) Results.—The load P_1 must be considered as it produces stresses in the truss. If R_2 coincides with the top chord there will be no stresses in the other members of the truss on the leeward side; if line of action of R_2 passes outside and above the truss the lower chord will be in tension; while if the line of action is below the upper chord the lower chord will be in compression. These statements may be checked by taking moments about the upper peak of the truss. It will be seen in Problems 11, 12, and 13 that there will be no reversal of stress when the dead load and wind load stresses are combined. This is commonly true for simple Fink trusses resting on walls; but is not true for Fink trusses supported on columns, nor is it always true for Fink trusses simply supported.

PROBLEM 13a. WIND LOAD STRESSES IN A FINK TRUSS—ROLLERS ON THE WINDWARD SIDE—BY GRAPHIC RESOLUTION.

(a) Problem.—Given the same truss and wind load as in Problem 12a, and with rollers under the windward side. Calculate the reactions by means of force and equilibrium polygons. Calculate the stresses by graphic resolution. Scale of truss, 1″ = 10′-0″. Scale of loads, 1″ = 3000 lbs.


Graphic Statics
Problem 13.
Same truss as in Problem 11.
Rollers windward
Scale 1" = 10'
0' 5' 10'
P1
P2
P3
P4
P5
R1 = 7000
R2 = 8400
60'-0"
0# 3000# 6000#
Scale 1" = 3000#
Horizontal Wind Load
= 30# per sq. ft.
Normal Component
= 26# per sq. ft.
P = 14 x √(20²+30²)/4 x 26 = 3300#

PROBLEM 14. WIND AND CEILING LOAD STRESSES IN AN UNSYMMETRICAL TRUSS, BY GRAPHIC RESOLUTION.

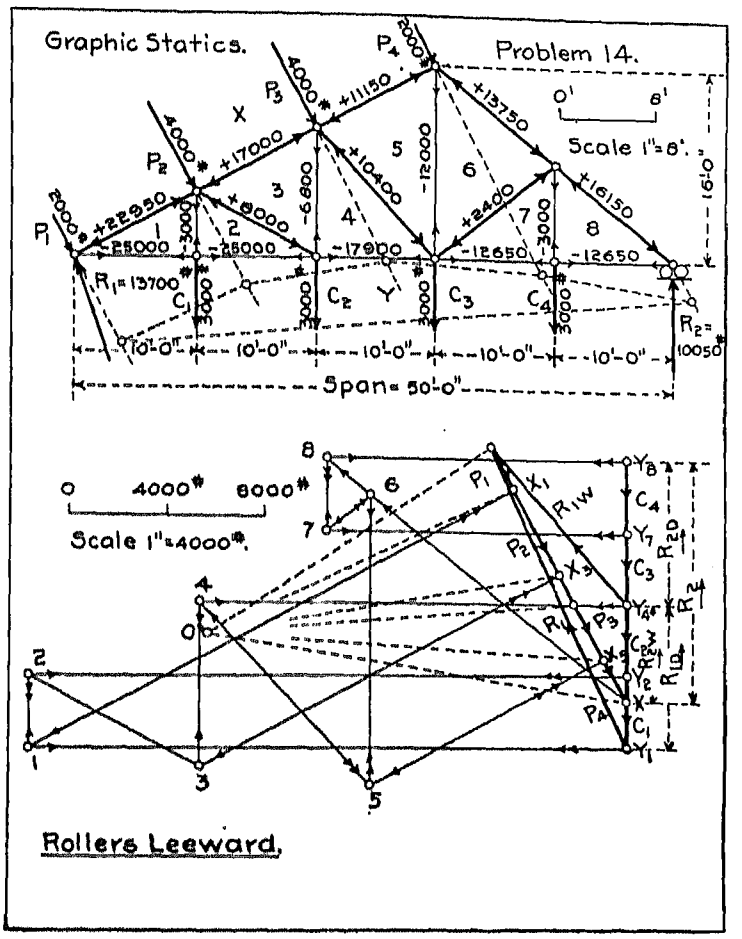
(a) Problem.—Given the unsymmetrical triangular truss, span 50′–0″, height 16′–0″, rollers leeward, wind joint load 4000 lbs., ceiling joint load 3000 lbs. Calculate the stresses due to both systems of loading by graphic resolution. Scale of truss, 1″ = 8′–0″. Scale of loads, 1″ = 4000 lbs.

(b) Methods.—Calculate the reactions due to the wind loads, using the method of Fig. 33. Calculate the reactions due to ceiling loads. Then place the y point of the wind load line on the point separating the ceiling load reactions (the x point). The wind loads are laid off in order downwards, while the ceiling loads are laid off in order upwards. The left reaction, R_1, is the resultant of R_{1W} and R_{1D} while the right reaction, R_2, is the resultant of R_{2W} and R_{2D}. The calculation of the stresses is begun at R_1, as in Problem 10. The stresses are then calculated by passing to joint C_1, then to joint P_2, then to C_2, etc., until the stress diagram is checked up at R_2.

(c) Results.—The members 1–2 and 7–8 are simply hangers to carry the ceiling loads. The triangular truss in this problem is of the Howe type, the verticals being in tension, while the diagonal web members are in compression. This truss is expensive to build of iron or steel but is quite a satisfactory type where iron is expensive and wood is cheap, and is used for the struts.

PROBLEM 14a. WIND AND CEILING LOAD STRESSES IN AN UNSYMMETRICAL TRUSS, BY GRAPHIC RESOLUTION.

(a) Problem.—Given the unsymmetrical triangular truss span 50′–0″, height, 16′–0″, rollers windward, wind joint load 4000 lbs., ceiling joint load 3000 lbs. Calculate the stresses due to both systems of loading by graphic resolution. Scale of truss, 1″ = 8′–0″. Scale of loads, 1″ = 4000 lbs.


Graphic Statics.
Problem 14.
Scale 1"=8'.
Span = 50'-0"
Scale 1"=4000#.
Rollers Leeward.

PROBLEM 15. DEAD LOAD STRESSES IN A TRIANGULAR TRUSS BY GRAPHIC AND ALGEBRAIC MOMENTS.

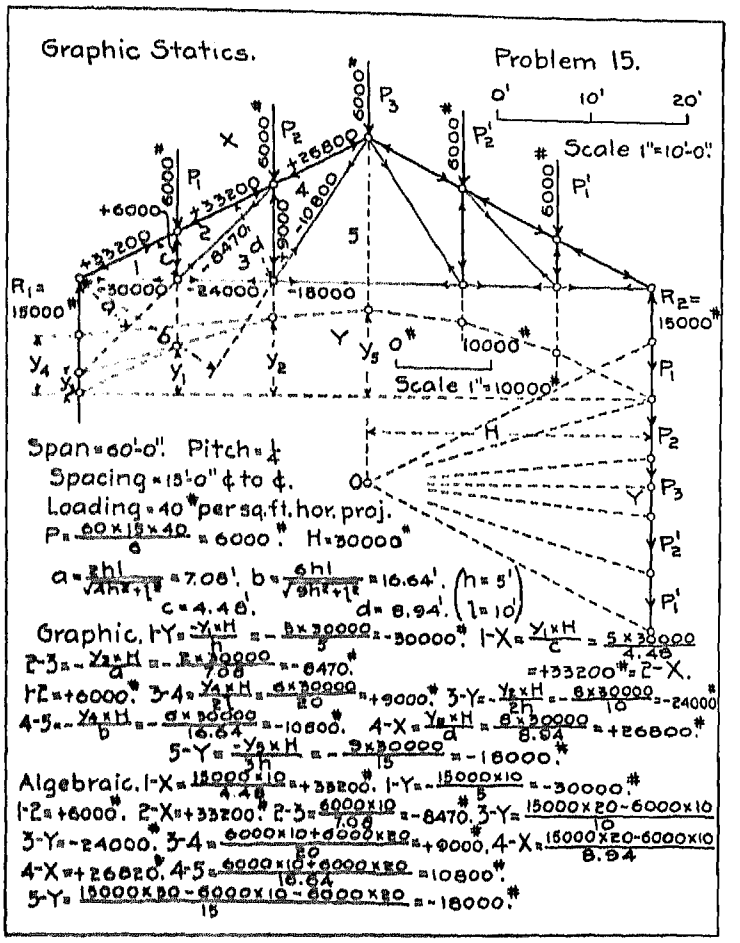
(a) Problem.—Given a triangular truss, span 60′–0″, pitch ¼, trusses spaced 15′–0″, dead load 40 lbs. per sq. ft. of horizontal projection. Calculate the stresses by graphic moments. Check by calculating the stresses by algebraic moments, giving all equations. Scale of truss, 1″ = 10′–0″. Scale of loads, 1″ = 10,000 lbs. Use pole distance $H = 30{,}000$ lbs.

(b) Methods.—Use the methods for algebraic and graphic moments described in Fig. 28 and Fig. 29, respectively. Calculate all moment arms and check by scaling from the diagram. The pole distance is measured in pounds, while the intercepts are measured to the same scale as the truss. Take the section and choose the center of moments so that but one unknown force will produce moments. Take the unknown external force as acting from the outside toward the cut section, the sign of the result if plus will indicate compression, if minus tension. Be careful to take forces on one side of the cut section only.

(c) Results.—The kinds of stress in the members are the same as in Problem 9. The center of moments used in calculating each stress can be easily determined from the equations. The method of algebraic moments is much used for calculating stresses in bridges, and other frameworks which carry moving loads. The method of graphic moments is used principally as an explanatory method.

PROBLEM 15a. DEAD LOAD STRESSES IN A TRIANGULAR TRUSS BY GRAPHIC AND ALGEBRAIC MOMENTS.

(a) Problem.—Given a triangular truss, span 60′–0″, pitch ⅓, trusses spaced 16′–0″, dead lead 40 lbs. per sq. ft. of horizontal projection. Calculate the stresses by graphic moments. Check by calculating the stresses by algebraic moments, giving all equations. Scale of truss, 1″ = 10′–0″. Scale of loads, 1″ = 10,000 lbs. Use pole distance $H = 30{,}000$ lbs.


Graphic Statics.
Problem 15.
Scale 1"=10'-0"
Scale 1"=10000#
Span=60'-0". Pitch=1/4
Spacing=15'-0" ¢ to ¢.
Loading=40# per sq.ft. hor. proj.
P=60×15×40/6=6000#. H=30000#
a=2hl/√4h²+l²=7.08'. b=6hl/√9h²+l²=16.64'. (h=5' l=10')
c=4.48'. d=8.94'.
Graphic. 1-Y=-Y1×H/h=-6×30000/5=-30000#. 1-X=Y1×H/c=5×30000/4.48=+33200#=2-X.
2-3=-Y2×H/a=-8×30000/7.08=-8470#.
1-2=+6000#. 3-4=Y4×H/2h=6×30000/10=+9000#. 3-Y=-Y2×H/2h=-8×30000/10=-24000#
4-5=-Y4×H/b=-6×30000/16.64=-10800#. 4-X=Y4×H/d=6×30000/8.94=+26800#
5-Y=-Y3×H/3h=-9×30000/15=-18000#.
Algebraic. 1-X=15000×10/4.48=+33200#. 1-Y=-15000×10/5=-30000#.
1-2=+6000#. 2-X=+33200#. 2-3=6000×10/7.08=-8470#. 3-Y=15000×20-6000×10/10
3-Y=-24000#. 3-4=6000×10+6000×20/20=+9000#. 4-X=15000×20-6000×10/8.94
4-X=+26820#. 4-5=6000×10+6000×20/16.64=10800#.
5-Y=15000×20-6000×10-6000×20/15=-18000#.

PROBLEM 16. DEAD LOAD STRESSES IN A CANTILEVER TRUSS BY GRAPHIC RESOLUTION.

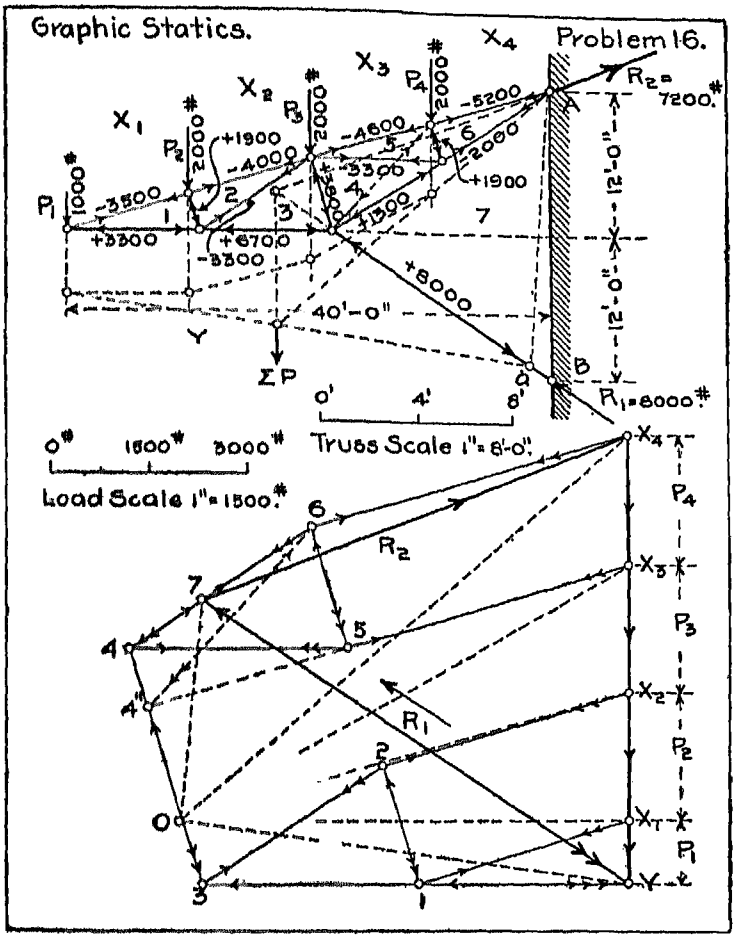
(a) Problem.—Given the Fink cantilever truss, span 40′–0″, depth 12′–0″, joint load 2000 lbs. Calculate the reactions by means of force and equilibrium polygons. Calculate the stresses by graphic resolution. Scale of truss, 1″ = 8′–0″. Scale of loads, 1″ = 1500 lbs.

(b) Methods.—Calculate the reactions as in Fig. 16, page 30, noting that the loads in the two cases are laid off in different order. Note that the two reactions and the resultant of the external loads meet in a point, and that the reactions can be determined by means of this principle. The stress diagram is started with the load P_1 and is closed at R_1 and R_2.

(c) Results.—The upper chord is in tension, while the bottom chord is in compression, which is the reverse of conditions in simple trusses. In calculating the stresses due to wind load in this truss, the reaction R_1 will be in the line of 7–4, and R_2 will pass through A, as in the case of dead loads. The resultant of the wind loads and the two reactions will meet in a point, and the solution is essentially the same as for dead loads. It should be noted that the closing line of the equilibrium polygon, A–a, has its ends on the line of action of the resultants R_1 and R_2.

PROBLEM 16a. DEAD LOAD STRESSES IN A CANTILEVER TRUSS BY GRAPHIC RESOLUTION.

(a) Problem.—Given the Fink cantilever truss, span 50′–0″, depth 15′–0″, joint load 2500 lbs. Calculate the reactions by means of force and equilibrium polygons. Calculate the stresses by graphic resolution. Scale of truss, 1″ = 8′–0″. Scale of loads, 1″ = 2000 lbs.


Graphic Statics.
Problem 16.
X1
X2
X3
X4
P1
P2
P3
P4
1000#
2000#
2000#
2000#
-3500
+3500
+1900
-4000
+6700
-3500
-4600
-5200
+3300
+1300
+1900
2000
+8000
40'-0"
12'-0"
12'-0"
R2= 7200.#
R1=8000.#
A
B
Y
ΣP
0'
4'
8'
0#
1500#
3000#
Truss Scale 1"=8'-0"
Load Scale 1"=1500.#
R2
R1
0
1
2
3
4
4'
5
6
7

PROBLEM 17. CALCULATION OF THE DEFLECTION OF A STEEL BEAM BY GRAPHICS.

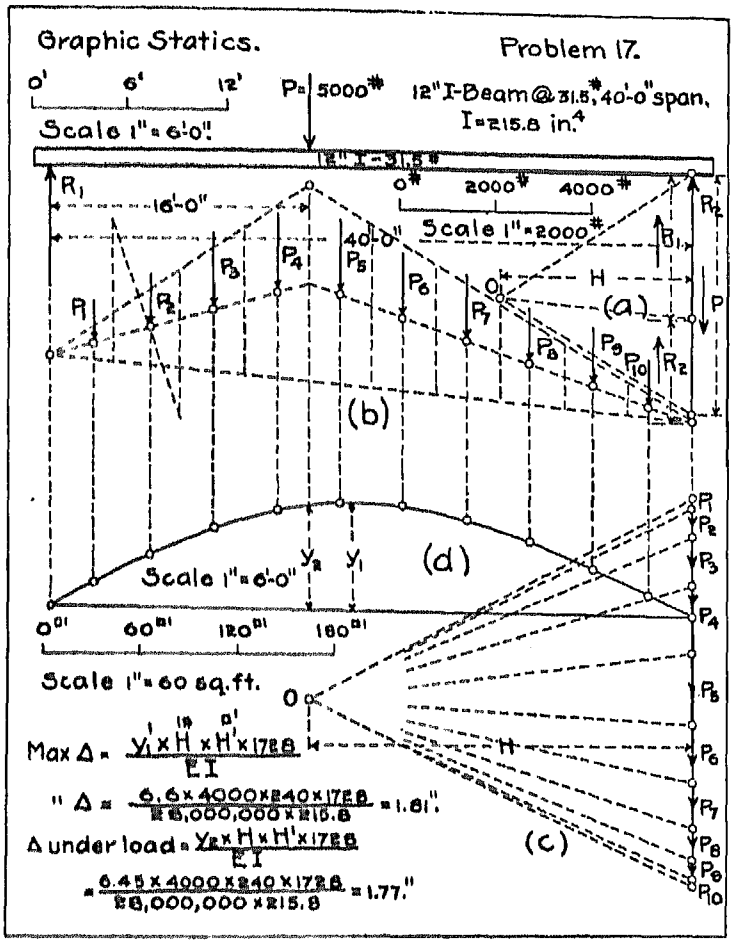
(a) Problem.—Given a 12″ I @ 31½ lbs. per foot, span 40′–0″, load 5000 lbs. applied 16′–0″ from the left support. $I = 215.8$ in.^{4}. $E = 28{,}000{,}000$. Calculate the maximum deflection due to the load, and the maximum deflection under the load by the graphic method. Scale of beam, 1″ = 6′–0″. Scale of loads, 1″ = 2000 lbs. Pole distance, $H = 4000$ lbs. Scale of areas, 1″ = 60 sq. ft. Pole distance, $H' = 240$ sq. ft.

(b) Methods.—Construct force polygon (a) and draw bending-moment polygon (b). Divide polygon (b) into segments, and assume that each area acts as a load through its center of gravity. Construct force polygon (c), and draw equilibrium polygon (d). Polygon (d) is a curve which has ordinates proportional to the true deflections.

(c) Results.—The maximum deflection comes between the load and the center of the beam. If the area of the polygon (b) was measured in square inches and the ordinates in (d) measured in inches the deflection would be $\Delta = y \times H \times H' \div E\,I$. In the problem this result must be multiplied by 1728. The closing lines of polygons (b) and (d) need not be horizontal. The solution given above may be very simply stated as follows: Construct the bending-moment polygon for the given loading on the beam. Load the beam with this bending-moment polygon, and with a force polygon having a pole distance equal to $E\,I$, construct an equilibrium polygon; this polygon will be the elastic curve of the beam. It is not commonly convenient to use a pole distance equal to $E\,I$, and a pole distance H is used, where $n\,H$ equals $E\,I$. For a discussion of this subject see Chapter XVa.

PROBLEM 17a. CALCULATION OF THE DEFLECTION OF A STEEL BEAM BY GRAPHICS.

(a) Problem.—Given a 12″ I @ 31½ lbs. per foot span 40′–0″, load 3000 lbs. applied 16′–0″ from the left support, and 3000 lbs. applied 12′–0″ from the right support. $I = 215.8$ in.^{4}. $E = 28{,}000{,}000$. Calculate the maximum deflection due to the load, and the maximum deflection under the load by the graphic method. Scale of



beam, 1" = 6'–0". Scale of loads, 1" = 2000 lbs. Pole distance, H = 4000 lbs. Scale of areas, 1" = 60 sq. ft. Pole distance, H' = 240 sq. ft.

PROBLEM 18. DEAD LOAD STRESSES IN A WARREN TRUSS BY GRAPHIC RESOLUTION.

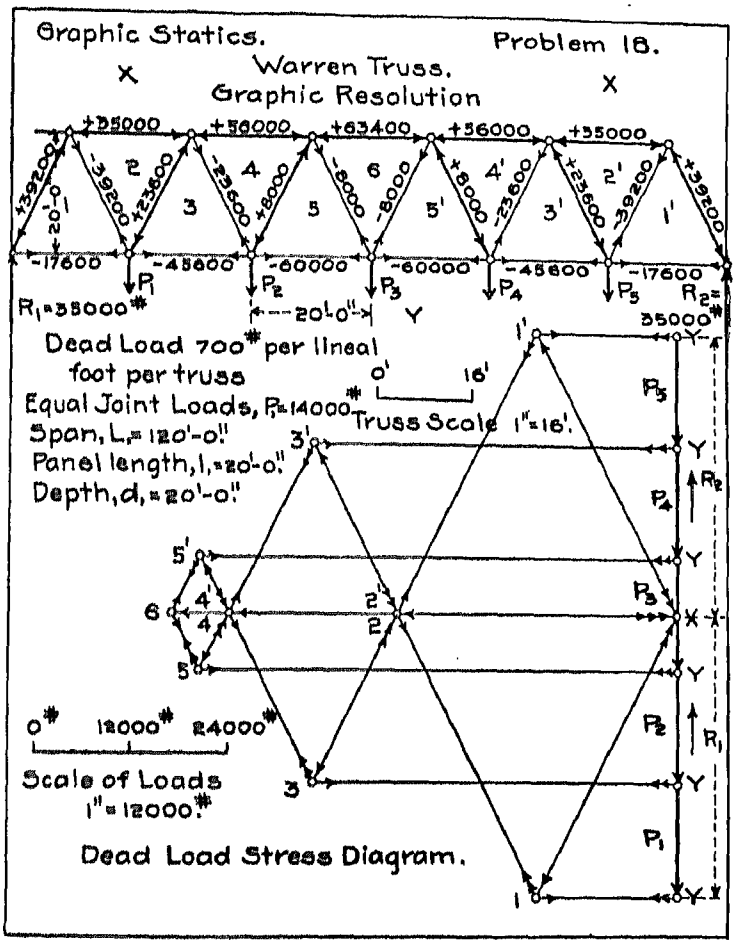
(a) Problem.—Given a Warren truss, span 120′–0″, panel length 20′–0″, depth 20′–0″, dead load 700 lbs. per ft. per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″–16′–0″. Scale of loads, 1″ = 12,000 lbs.

(b) Methods.—The loads beginning with the first load on the left are laid off from the bottom upwards. The calculation of the stresses is started at the left reaction, and the stress diagram is closed at the right reaction. For additional information on the solution see page 70.

(c) Results.—The top chord is in compression, the bottom chord is in tension; all web members leaning toward the center of the truss are in compression, while the web members leaning toward the abutments are in tension. All web members meeting on the unloaded chord (top chord) have stresses equal in amount but opposite in sign. The stresses in the lower chord are the arithmetical means of the stresses in the top chord. The Warren truss is commonly made of iron or steel, the most common section for the members being two angles placed back to back.

PROBLEM 18a. DEAD LOAD SRESSES IN A WARREN TRUSS BY GRAPHIC RESOLUTION.

(a) Problem.—Given a Warren truss, span 126′–0″, panel length 18′–0″, depth 20′–0″, dead load 700 lbs. per ft. per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 15′–0″. Scale of loads, 1″ = 12,000 lbs.


Graphic Statics.
Problem 18.
Warren Truss.
Graphic Resolution
+25000
+56000
+63400
+56000
+35000
+39200
-39200
+23600
-23600
+8000
-8000
-8000
+8000
-23600
+23600
-39200
+39200
-17600
-45600
-60000
-60000
-45600
-17600
P1
P2
P3
P4
P5
R1=35000#
R2=35000#
20'-0"
Y
Dead Load 700# per lineal foot per truss
Equal Joint Loads, P=14000#
Span, L,= 120'-0"
Panel length, l,=20'-0"
Depth, d,=20'-0"
0'
16'
Truss Scale 1"=16'.
0#
12000#
24000#
Scale of Loads 1"=12000.#
Dead Load Stress Diagram.

PROBLEM 19. DEAD LOAD STRESSES IN A HOWE TRUSS BY GRAPHIC RESOLUTION.

(a) Problem.—Given a Howe truss, span 160′–0″, panel length 20′–0″, depth 24′–0″, dead load 600 lbs. per lineal foot of truss. Calculate the dead load stresses by graphic resolution.. Scale of truss, 1″ = 25′–0″. Scale of loads, 1″ = 15,000 lbs.

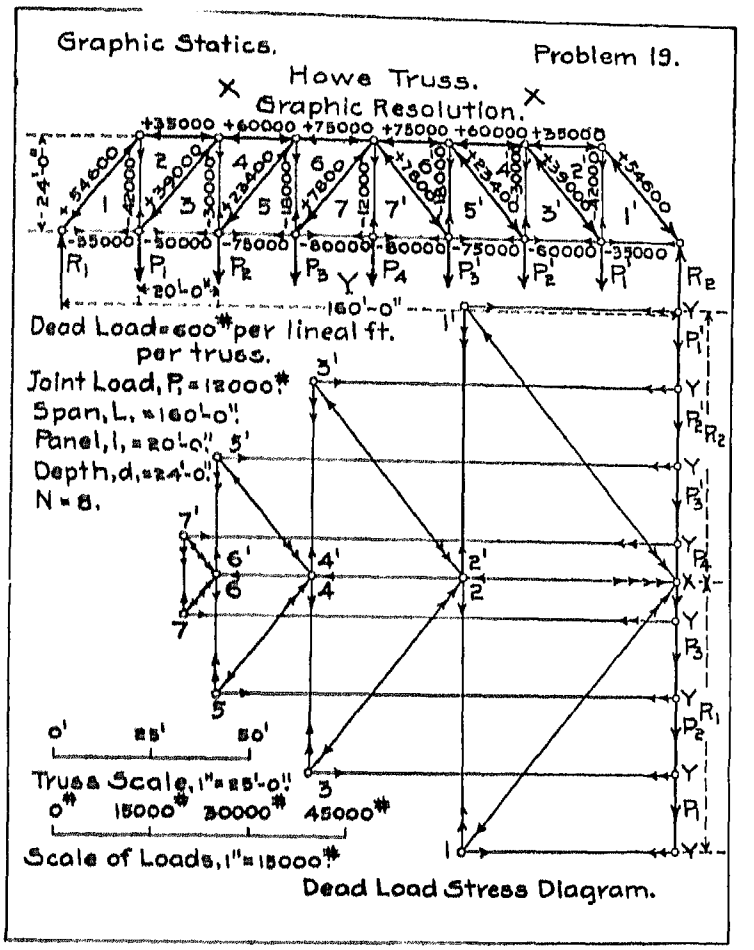
(b) Methods.—The loads beginning with the first load on the left are laid off from the bottom upwards. Calculate the stresses by graphic resolution, beginning at R_1 and checking at R_2, following the order shown in the stress diagram.

(c) Results.—The top chord is in compression and the bottom chord is in tension as in the Warren truss. All inclined web members are in compression, while all vertical web members are in tension. The stresses in the verticals are equal to the vertical components of the diagonal members meeting them on the unloaded chord. Stresses in certain panels in top and bottom chords are equal.

The Howe truss is commonly built with timber upper and lower chords and diagonal struts, the only iron being the vertical ties and cast iron angle blocks to take the bearing of the timber struts. This makes a very satisfactory truss and is quite economical where timber is cheap.

PROBLEM 19a. DEAD LOAD STRESSES IN A HOWE TRUSS BY GRAPHIC RESOLUTION.

(a) Problem.—Given a Howe truss, span 162′–0″, panel length 18′–0″, depth 24′–0″, dead load 600 lbs. per lineal foot of truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 25′–0″. Scale of loads, 1″ = 15,000 lbs.


Graphic Statics. Problem 19.
Howe Truss.
Graphic Resolution.
+35000 +60000 +75000 +75000 +60000 +35000
-35000 -50000 -75000 -80000 -80000 -75000 -60000 -35000
R1 P1 P2 P3 P4 P3' P2' P1' R2
160'-0"
Dead Load=600# per lineal ft. per truss.
Joint Load, P=12000#
Span, L.=160'-0"
Panel, l.=20'-0"
Depth, d.=24'-0"
N=8.
0' 25' 50'
Truss Scale, 1"=25'-0"
0# 15000# 30000# 45000#
Scale of Loads, 1"=15000#
Dead Load Stress Diagram.

PROBLEM 20. DEAD LOAD STRESSES IN A PRATT TRUSS BY GRAPHIC RESOLUTION.

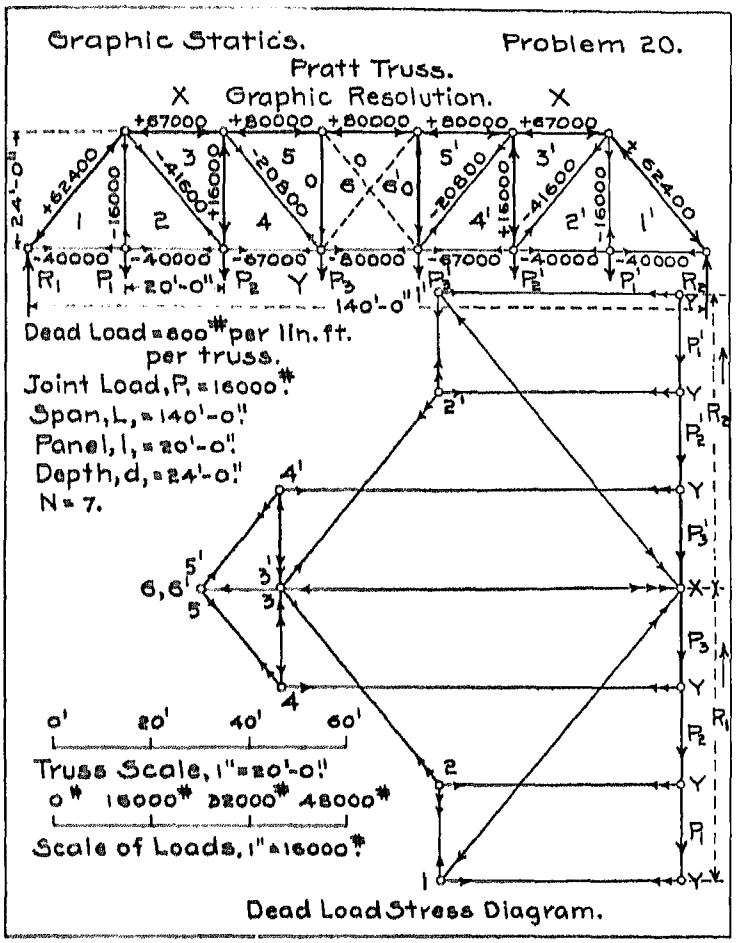
(a) Problem.—Given a Pratt truss, span 140′–0″, panel length 20′–0″, depth 24′–0″, dead load 800 lbs. per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss 1″ = 20′–0″. Scale of loads, 1″ = 16,000 lbs.

(b) Methods.—The loads beginning with the first load on the left are laid off from the bottom upwards. Calculate the stresses by graphic resolution, beginning at R_1 and checking at R_2, following the order shown in the stress diagram.

(c) Results.—The top chord is in compression and the bottom chord is in tension as in the Warren and Howe trusses. The inclined web members are in tension, while the vertical posts are in compression. Member 1–2 is simply a hanger. There is no stress due to dead loads in the diagonal members in the middle panel. The stresses in the posts are equal to the vertical components of the diagonal members meeting them on the unloaded chord. Stresses in certain panels in the top and bottom chord are equal. The Pratt truss is quite generally used for steel bridges, and is also used for combination bridges, where the tension members are made of iron or steel and the compression members are made of timber.

PROBLEM 20a. DEAD LOAD STRESSES IN A PRATT TRUSS BY GRAPHIC RESOLUTION.

(a) Problem.—Given a Pratt truss, span 160′–0″, panel length 20′–0″, depth 24′–0″, dead load 800 lbs. per lineal feet per truss. Calculate the dead load stresses by graphic resolution. Scale of truss 1″ = 25′–0″. Scale of loads, 1″ = 20,000 lbs.


Graphic Statics. Problem 20.
Pratt Truss.
X Graphic Resolution. X
+67000 +80000 +80000 +80000 +67000
+62400 -41600 -20800 0 -20800 -41600 +62400
+16000 +16000 +16000 +16000
24'-0"
1 2 3 4 5 6 6' 5' 4' 3' 2' 1'
-40000 -40000 -67000 -80000 -67000 -40000 -40000
R1 P1 P2 Y P3 P3' P2' P1' R2
20'-0"
140'-0"
Dead Load = 800# per lin. ft. per truss.
Joint Load, P = 16000#
Span, L, = 140'-0"
Panel, l, = 20'-0"
Depth, d, = 24'-0"
N = 7.
0' 20' 40' 60'
Truss Scale, 1" = 20'-0"
0# 16000# 32000# 48000#
Scale of Loads, 1" = 16000#
Dead Load Stress Diagram.

PROBLEM 21. DEAD LOAD STRESSES IN A CAMEL-BACK TRUSS BY GRAPHIC RESOLUTION.

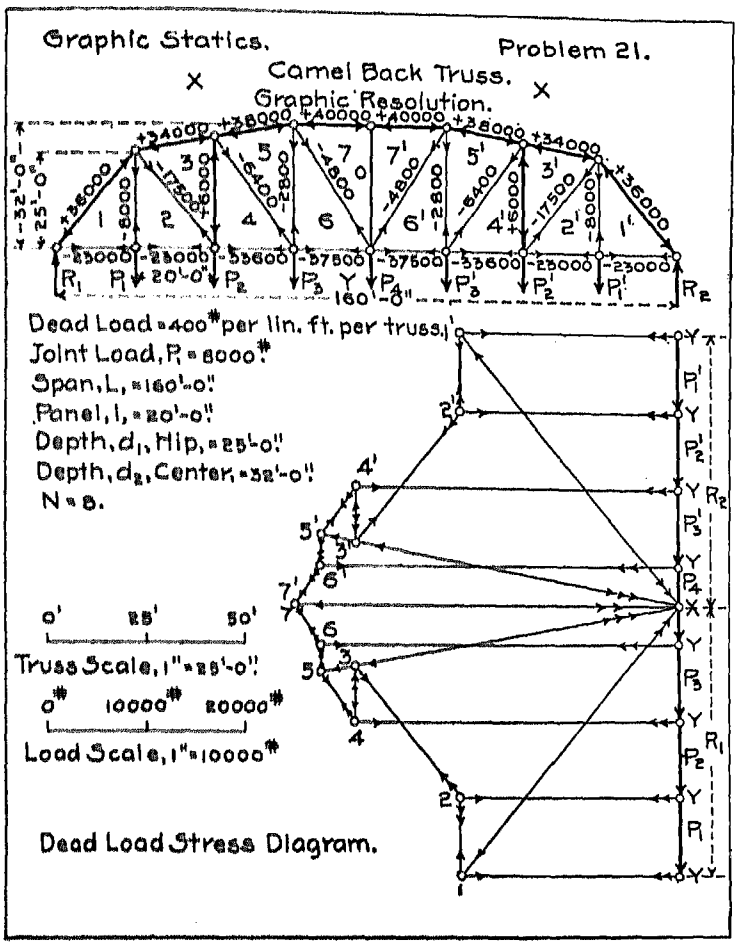
(a) **Problem.**—Given a camel-back (inclined Pratt) truss, span 160′–0″, panel length 20′–0″, depth at the hip 25′–0″, depth at the center 32′–0″, dead load 400 lbs. per lineal foot per truss. Calculate dead load stresses by graphic resolution. Scale of truss, 1″ = 25′–0″. Scale of loads, 1″ = 10,000 lbs.

(b) **Methods.**—The loads beginning with the first load on the left are laid off from the bottom upwards. Calculate the stresses by graphic resolution, beginning at R_1 and checking at R_2. Follow the order given in the stress diagram.

(c) **Results.**—The top chord is in compression and the bottom chord is in tension. All inclined web members are in tension; while part of the posts are in tension and part are in compression. Member 1–2 is simply a hanger and is always in tension. This type of truss is quite generally used for steel and combination bridges for spans from 150 feet to 200 feet, and also for roof trusses for long span, where it is loaded on the top chord and bottom chord, or on the top chord alone.

PROBLEM 21a. DEAD LOAD STRESSES IN A CAMEL-BACK TRUSS BY GRAPHIC RESOLUTION.

(a) **Problem.**—Given a camel-back (inclined Pratt) truss, span 180′–0″, panel length 20′–0″ (three panels with parallel chords), depth at the hip 25′–0″, depth at the center 32′–0″, dead load 400 lbs. per lineal foot per truss. Calculate dead load stresses by graphic resolution. Scale of truss, 1″ = 25′–0″. Scale of loads, 1″ = 12,000 lbs.


Graphic Statics. Problem 21.
Camel Back Truss.
Graphic Resolution.
Dead Load = 400# per lin. ft. per truss.
Joint Load, P = 8000#.
Span, L, = 160'-0".
Panel, l, = 20'-0".
Depth, d_1, Hip, = 25'-0".
Depth, d_2, Center, = 32'-0".
N = 8.
0' 25' 50'
Truss Scale, 1" = 25'-0".
0# 10000# 20000#
Load Scale, 1" = 10000#
Dead Load Stress Diagram.

PROBLEM 22. WIND LOAD STRESSES IN A TRESTLE BENT.

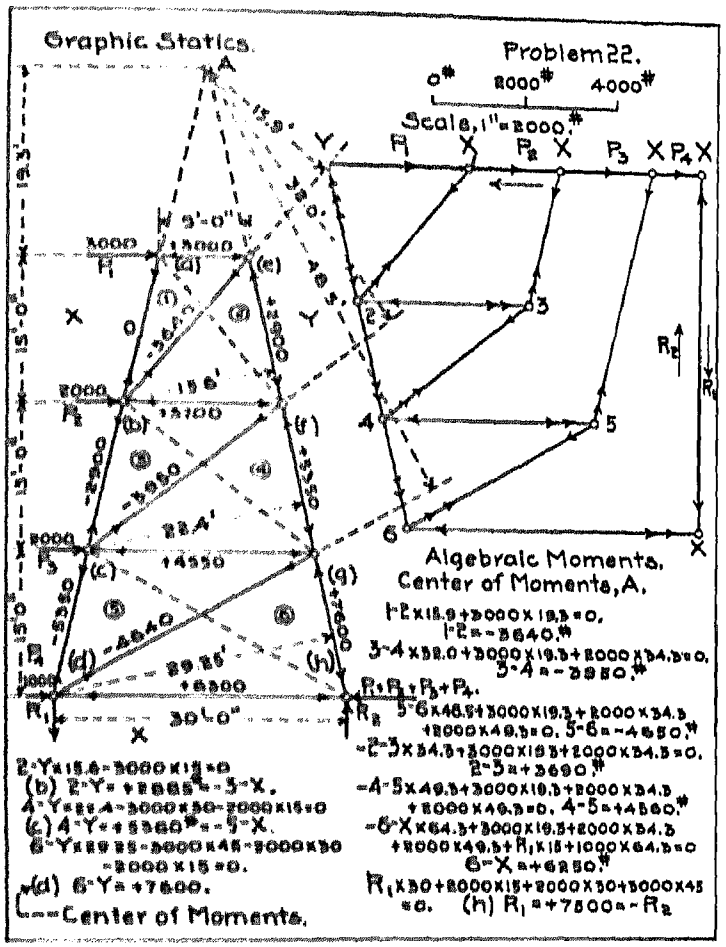
(a) **Problem.**—Given a trestle bent, height 45′–0″, width at the base 30′–0″, width at the top 9′–0″, wind loads P_1, P_2, P_3, P_4, as shown. Calculate the stresses in the members of the bent due to wind loads by algebraic moments and check by calculating the stresses by graphic resolution. Assume the diagonal members to be tension members, and that the dotted members shown are not acting. Scale of truss, 1″ = 10′–0″. Scale of loads, 1″ = 2000 lbs.

(b) **Methods.**—(1) Algebraic Moments: To calculate the stresses in the diagonal members take center of moments about the point A, the point of intersection of the inclined posts. Then to calculate stress in 1–2, take section cutting 1–2, 1–X and 2–Y; assume the external force as acting from the outside toward the cut section, and we have $1\text{–}2 \times 15.9 + 3000 \times 19.3 = 0$ and $1\text{–}2 = -3640$ lbs. Stresses in 3–4, 5–6, 2–3, 4–5 and 6–X are found in a similar manner. To obtain reaction R_1 take moments about (h), and $R_1 \times 30 + 2000 \times 15 + 2000 \times 30 + 3000 \times 45 = 0$ and $R_1 = +7500$ lbs. $= -R_2$. To calculate stresses in 2–Y, 4–Y and 6–Y, take moments about (b), (c) and (d), respectively. To calculate stresses in 3–X and 5–X, take moments about (f) and (g), respectively. (2) Graphic Resolution: The loads P_1, P_2, P_3 and P_4 are laid off horizontally as shown, and with load P_1 at (c) the stress triangle Y–2–4 is drawn. The remainder of the solution can be easily followed.

(c) **Results.**—For the reason that the wind may blow from the opposite direction, both sets of stresses must be considered in designing each leg.

PROBLEM 22a. WIND LOAD STRESSES IN A TRESTLE BENT.

(a) **Problem.**—Given a trestle bent, height 45′–0″, width at the base 30′–0″, width at the top 9′–0″, wind loads P_1, P_2, P_3, P_4, as shown, and $P_0 = 3000$ lbs. acting 8′–0″ above top of trestle. Calculate the stresses in the members of the bent due to wind loads by algebraic moments and check by calculating the stresses by graphic resolution. Assume the diagonal members to be tension members, and that the dotted members shown are not acting. In solving the stresses by graphic resolution continue the posts up to the line of P_0 and substitute an auxiliary panel to transfer P_0 to the bent. Scale of truss, 1″ = 10′–0″. Scale of loads, 1″ = 3000 lbs.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Center of Moments.

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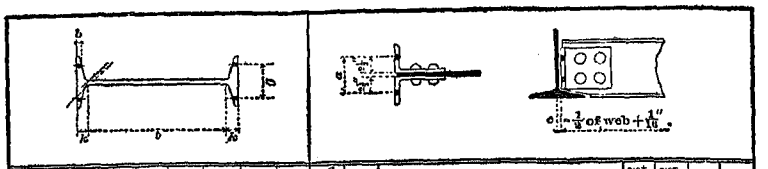
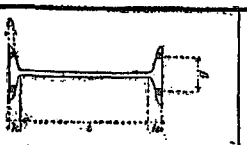
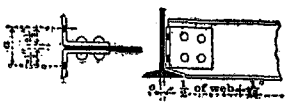
CARNEGIE I-BEAMS.



I	WEIGHT PER FOOT	FLANGE	WEB	GAUGE	TANG'T	DIST.	GRIP	MAX. RIVET OR BOLT	WALL BEARING	WALL PL	STANDARD FRAMING	DIST.	DIST.	WEIGHT PER FOOT	I
				g	t	k	b					a	c		
INCHES	POUNDS	INCHES	INCHES	INCHES	INCHES	INCHES	INCHES	INCHES	INCHES			INCHES	INCHES	POUNDS	INCHES
24	100.0	7 1/4	3/4	4	20 3/4	1 5/8	5/8	1	16	16" x 1" x 1'-4" 73#	24" 2-15 4"x 4"x 7/16 x 1'-0" Wt. 37#	5 0/4	7/16	100.0	24
	95.0	7 5/16	21/16	"	"	"	"					5 11/16	7/16	95.0	
	90.0	7 1/4	5/8	"	"	"	"					5 5/8	5/8	90.0	
	85.0	7 1/16	9/16	"	"	"	"					5 9/16	5/8	85.0	
	80.0	7.	1/2	"	"	"	"					5 1/2	5/8	80.0	
20	100.0	7 29/32	7/8	"	16 1/2	1 1/4	15/16	7/8	16	16" x 1" x 1'-4" 73#	18" and 20" 2-15 4"x 4"x 7/16 x 1'-0" Wt. 37#	5 7/8	1/2	100.0	20
	95.0	7 27/32	15/16	"	"	"	"					5 15/16	1/2	95.0	
	90.0	7 13/16	25/32	"	"	"	"					5 3/4	7/16	90.0	
	85.0	7 3/4	25/32	"	"	"	"					5 11/16	7/16	85.0	
	80.0	7	13/32	"	"	"	"					5 5/8	5/8	80.0	
	75.0	6 31/32	25/32	"	17	1 1/2	3/4					5 15/16	7/16	75.0	
	70.0	6 7/8	3/8	"	"	"	"					5 5/16	0/0	70.0	
	65.0	6 13/16	3/8	"	"	"	"					5 1/4	5/16	65.0	
18	70.0	6 1/4	25/32	3 3/4	15 1/4	1 3/8	5/8	7/8	16	16" x 1" x 1'-4" 73#	18" and 20" 2-15 4"x 4"x 7/16 x 1'-0" Wt. 31#	5 0/4	7/16	70.0	18
	65.0	6 1/16	3/8	"	"	"	"					5 5/8	5/8	65.0	
	60.0	6 3/16	11/32	"	"	"	"					5 5/16	5/8	60.0	
	55.0	6	7/16	"	"	"	"					5 1/2	5/16	55.0	
15	200.0	6 25/32	1 1/16	"	11	2	1	7/8	12	12" x 3/4" x 1'-4" 4#	15" 2-15 0"x 4"x 7/16 x 0'-10" Wt. 27#	6 0/16	11/16	100.0	15
	95.0	6 23/32	1 1/16	"	"	"	"					6 1/16	5/8	95.0	
	90.0	6 3/4	31/32	"	"	"	"					6	0/16	90.0	
	85.0	6 15/32	7/8	"	"	"	"					6 7/8	1/2	85.0	
	80.0	6 13/32	15/16	"	"	"	"					5 13/16	1/2	80.0	
	75.0	6 5/32	7/8	"	11 1/2	1 5/8	1/4					5 3/4	1/2	75.0	
	70.0	6 1/16	29/32	"	"	"	"					5 3/4	7/16	70.0	
	65.0	6 1/32	29/32	"	"	"	"					5 11/16	7/16	65.0	
	60.0	6	25/32	"	"	"	"					5 5/8	5/8	60.0	
	55.0	5 7/8	31/32	3 1/2	12 1/2	1 1/4	5/16	3/4				5 0/8	3/8	55.0	
	50.0	5 3/4	13/32	"	"	"	"					5 0/16	5/8	50.0	
	45.0	5 11/16	7/16	"	"	"	"					5 1/8	5/16	45.0	
	42.0	5 1/2	15/32	"	"	"	"					5 7/16	1/4	42.0	
12	55.0	5 5/8	1/2	3 1/4	9 1/2	1 5/8	11/16	3/8	12	12" x 3/4" x 1'-0" 31#	12" 2-12 0"x 4"x 7/16 x 0'-7 1/2" Wt. 20#	5 12/16	1/2	55.0	12
	50.0	5 1/4	11/16	"	"	"	"					5 11/16	7/16	50.0	
	45.0	5 0/4	0/16	3	"	"	"					5 0/16	0/8	45.0	
	40.0	5 1/4	0/16	"	"	"	"					5 1/8	5/16	40.0	
	35.0	5 3/8	7/16	"	9 1/2	1 1/2	5/8					5 7/16	6/16	35.0	
	31.5	5	15/32	"	"	"	"					5 0/8	1/4	31.5	

CARNEGIE I-BEAMS.



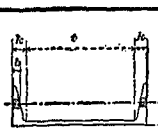


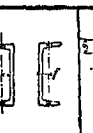
I	WEIGHT PER FOOT	FLANGE	WEB	GAUGE	TARGET	DIST.	GRIP	MAX. RIVET OR BOLT	WALL BEARING	WALL PL.	STANDARD FRAMING	DIST.	DIST.	WEIGHT PER FOOT	I
INCHES	POUNDS	INCHES	INCHES	g INCHES	t INCHES	k INCHES	b INCHES	INCHES	INCHES			a INCHES	c INCHES	POUNDS	INCHES
10	40.0	6 25/32	1/2	3	8	1	7/16			17#		6 3/4	7/16	40.0	
	35.0	4 15/16	13/32	2 1/2	"	"	"	3/4	8			5 7/8	3/8	35.0	10
	30.0	4 31/32	19/64	"	"	"	"					5 5/8	5/16	30.0	
	25.0	4 33/64	1/4	"	"	"	"			8" x 8" x 1'-0"		5 3/8	1/4	25.0	
9	35.0	4 1/2	29/64	"	7	"	"					5 3/4	7/16	35.0	
	30.0	4 15/32	9/32	"	"	"	"					5 5/8	5/16	30.0	
	25.0	4 7/16	17/64	2 1/2	"	"	"	3/4	8		7", 8", 9", & 10"	5 7/16	1/4	25.0	9
	21.0	4 13	5/16	"	"	"	"					5 1/8	1/4	21.0	
8	25.5	4 1/4	15/32	"	6 1/2	7/8	"			12#		5 1/2	1/4	25.5	
	23.0	4 13	7/16	"	"	"	"					5 1/4	1/4	23.0	
	20.5	4 3/8	3/8	2 1/2	"	"	"	3/4	8	8" x 5/8" 0-8"	2-15 0" x 4" x 7/16" x 0'-5" Wt. 14#	5 1/8	1/4	20.5	8
	18.0	4	5/16	"	"	"	"					5	1/8	18.0	
7	20.0	3 7/8	13/32	"	5 1/2	"	5/8					5 1/2	1/4	20.0	
	17.5	3 3/4	11/32	"	"	"	"	5/8	8			5 3/8	1/4	17.5	7
	15.0	3 11/16	1/4	"	"	"	"					5 1/4	1/8	15.0	
6	17.25	3 9/16	15/32	2	4 1/2	3/4	"			5#		5 1/4	3/8	17.25	
	14.75	3 7/16	7/16	"	"	"	"	5/8	6			5 3/8	1/4	14.75	6
	12.25	3 5/16	5/16	"	"	"	"					5 1/4	1/8	12.25	
5	14.75	3 5/8	1/2	1 3/4	3 1/2	3/4	5/16					5 1/2	1/4	14.75	
	12.25	3 1/2	7/16	"	"	"	"	1/2	6		5" & 6"	5 3/8	1/4	12.25	5
	9.75	3	5/16	"	"	"	"				2-15 6" x 4" x 7/16" x 0'-3" for 6" Wt. 8#	5 1/4	1/8	9.75	
4	10.5	2 7/8	13/32	1 1/4	2 3/4	"	"			6" x 1/2" 0-6"	2-15 " " 0'-2 1/2" " 5" Wt. 7#	5 1/2	1/4	10.5	
	9.5	2 13/16	11/32	"	"	"	"	1/2	6			5 3/8	1/4	9.5	4
	8.5	2 25/32	1/4	"	"	"	"					5 1/4	1/8	8.5	
	7.5	2 23/32	1/4	"	"	"	"					5 1/8	1/8	7.5	
3	7.5	2 1/4	11/32	1 1/4	1 1/4	"	1/4				5" & 4"	5 3/8	1/4	7.5	
	6.5	2 3/16	1/4	"	"	"	"	1/2	6		2-15 6" x 4" x 7/16" x 0'-2" Wt. 6#	5 1/4	1/8	6.5	3
	5.5	2 1/8	3/16	"	"	"	"					5 1/8	1/8	5.5	

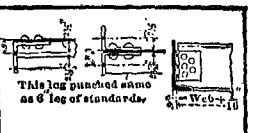
All rivets in standard framing angles are $\frac{5}{8}$" diam.
Weights of " " " include weight of shop rivets only.
When beams frame opposite each other into another beam with web thickness less than $\frac{5}{16}$" or where beams of short span lengths are loaded to their full capacity, it may be necessary to use framing angles of greater strength than the standards.
See table below for minimum span lengths.

I	WEIGHT	SPAN IN FT.	I	WEIGHT	SPAN IN FT.	I	WEIGHT	SPAN IN FT.	I	WEIGHT	SPAN IN FT.	I	WEIGHT	SPAN IN FT.	I	WEIGHT	SPAN IN FT.	I	WEIGHT	SPAN IN FT.
24	80.0	22.0				15	80.0	20.0							8	18.0	5.5	5	9.75	4.0
20	80.0	22.0	18	55.0	14.0	"	60.0	15.5	12	40.0	11.5	10	25.0	9.0	7	15.0	4.0	4	7.5	3.0
"	65.0	18.0				"	42.0	11.0	"	31.5	9.0	9	21.0	7.0	6	12.25	6.0	3	5.5	2.0

Carnegie Channels.







⌐	Weight per foot	Flange	Web	Gauge	Tang't.	Dist.	Grip	Max. rivet or bolt	Dist.	Gauge	Dist.	Dist.	Weight per foot	⌐
				g	t	k	b		d	f	s	c		
15	55.00	3 15/16	13/16	2 1/2	12 1/2	1 1/8	5/8	3/4	1 5/8	2 7/16	3 5/16	3/4	55.00	15
	50.00	3 3/4	3/4	"	"	"	"		"	2 3/8	3 1/4	5/8	50.00	
	45.00	3 5/8	5/8	2	"	"	"		"	2 5/16	3 1/8	1/2	45.00	
	40.00	3 1/2	1/2	"	"	"	"		"	2 1/8	3	3/8	40.00	
	35.00	3 3/8	3/8	"	"	"	"		"	2 1/16	2 15/16	1/4	35.00	
	33.00	3 5/16	5/16	"	"	"	"		"	2	2 15/16	1/4	33.00	
12	40.00	3 15/16	7/16	"	10	1	1/2	3/4	1 13/16	2 5/16	3 1/4	7/16	40.00	12
	35.00	3 7/8	3/8	"	"	"	"		"	2 3/16	3 1/4	1/2	35.00	
	30.00	3 7/8	3/8	1 1/2	"	"	"		"	1 15/16	3	3/8	30.00	
	25.00	3 7/8	3/8	"	"	"	"		"	1 3/4	2 15/16	5/16	25.00	
	20.50	2 15/16	5/16	"	"	"	"		"	1 1/2	2 15/16	1/8	20.50	
10	35.00	3 5/8	1/2	"	8 1/2	7/8	"	3/4	1 1/2	2 1/16	3 1/16	3/8	35.00	10
	30.00	3 5/8	1/2	"	"	"	"		"	1 15/16	3 1/16	1/2	30.00	
	25.00	3 1/2	3/8	"	"	"	7/8		"	1 13/16	3 1/16	3/8	25.00	
	20.00	3 1/16	5/16	1 1/2	"	"	"		"	1 5/8	2 7/8	5/16	20.00	
	15.00	2 11/16	1/4	"	"	"	"		"	1 1/2	2 7/8	1/8	15.00	
9	25.00	3 1/16	5/8	"	7 1/2	7/8	"	3/4	1 5/8	1 1/2	3 1/16	7/16	25.00	9
	20.00	2 15/16	7/16	"	"	"	1/2		"	1 7/16	2 15/16	3/8	20.00	
	15.00	2 1/2	3/8	1 1/2	"	"	"		"	1 11/16	2 15/16	1/8	15.00	
	13.25	2 1/16	7/16	"	"	"	"		"	1 3/16	2 1/2	1/16	13.25	
8	21.25	2 5/8	1/2	1 1/2	6 1/2	"	7/8	3/4	1 1/8	1 13/16	3 1/16	3/8	21.25	8
	18.75	2 5/8	1/2	"	"	"	"		"	1 5/8	3	5/16	18.75	
	16.25	2 7/8	3/8	"	"	"	3/4		"	1 5/8	2 7/8	5/16	16.25	
	13.75	2 5/8	5/16	1 1/2	"	"	"		"	1 3/8	2 13/16	1/8	13.75	
	11.25	2 1/4	5/16	"	"	"	"		"	1 3/16	2 1/2	1/16	11.25	
7	19.75	2 1/4	7/16	1 1/2	5 1/2	7/8	"	3/4	1 1/16	1 11/16	3 1/8	11/16	19.75	7
	17.25	2 1/16	7/16	"	"	"	"		"	1 5/16	3 1/8	5/16	17.25	
	14.75	2 1/8	7/16	"	"	"	"	5/8	"	1 1/4	2 13/16	1/4	14.75	
	12.25	2 1/8	5/16	1 1/2	"	"	"		"	1 1/8	2 13/16	1/8	12.25	
	9.75	2 1/8	1/4	"	"	"	"		"	1 1/8	2 1/2	1/4	9.75	
6	15.50	2 1/8	7/16	"	4 1/2	"	"	5/8	1 1/16	1 1/4	3 1/16	1/2	15.50	6
	13.00	2 1/8	5/16	"	"	"	"		"	1 1/8	2 13/16	1/4	13.00	
	10.50	2 1/8	1/4	"	"	"	"		"	1 1/4	2 13/16	1/8	10.50	
	8.00	1 13/16	1/4	1 1/4	"	"	"		"	1 1/4	2 1/16	1/4	8.00	
5	11.50	2 1/16	3/8	"	3 1/2	3/4	7/8	1/2	7/8	1 13/16	3	3/8	11.50	5
	9.00	1 15/16	5/16	"	"	"	"		"	1 7/16	2 13/16	1/8	9.00	
	6.50	1 3/4	1/4	"	"	"	"		"	1 1/16	2 1/16	1/4	6.50	
4	7.25	1 13/16	5/16	1	2 1/2	3/4	"	1/2	3/4	1 1/16	2 1/16	3/8	7.25	4
	6.25	1 13/16	1/4	"	"	"	"		"	1	2 1/16	5/16	6.25	
	5.25	1 11/16	1/4	"	"	"	"		"	15/16	2 1/16	1/4	5.25	
3	6.00	1 7/8	3/8	3/4	1 1/2	3/4	1/2	1/2	5/8	1	2 1/16	7/16	6.00	3
	5.00	1 3/4	1/4	"	"	"	"		"	7/8	2 1/16	5/16	5.00	
	4.00	1 5/8	1/4	"	"	"	"		"	13/16	2 1/16	1/4	4.00	

Carnegie Z-Bars.

All dimensions in inches

Nominal Size	Thickness	Actual Size Flanges & Web	Weight per Foot	Area in Sq. Inches	Gauge a	Max. Rivets a	Max. Rivets a^1	Gauge a^1	Thickness	Nominal Size
3	1/4	2⅝ × 3 × 2⅝	9.7	1.97	1¾	1/2	1/2	1¼	1/4	3
	5/16	2½ × 3⅛ × 2½	8.4	2.48	"	"	"	"	5/16	
	3/8	2⅜ × 3 × 2⅜	9.7	2.86	"	"	"	"	3/8	
	7/16	2½ × 3⅛ × 2½	11.4	3.36	"	"	"	"	7/16	
	1/2	2⅜ × 3 × 2⅜	12.8	3.69	"	"	"	"	1/2	
	9/16	2½ × 3⅛ × 2½	14.3	4.18	"	"	"	"	9/16	
4	1/4	3⅛ × 4 × 3⅛	8.2	2.41	2	1/2	1/2	2	1/4	4
	5/16	3½ × 4⅛ × 3½	10.3	3.03	"	"	"	"	5/16	
	3/8	3⅜ × 4½ × 3⅜	12.4	3.66	"	"	"	"	3/8	
	7/16	3⅛ × 4 × 3⅛	13.2	4.05	"	"	"	"	7/16	
	1/2	3½ × 4⅛ × 3½	15.3	4.66	"	"	"	"	1/2	
	9/16	3⅜ × 4½ × 3⅜	17.9	5.27	"	"	"	"	9/16	
	5/8	3⅛ × 4 × 3⅛	18.9	5.55	"	"	"	"	5/8	
	11/16	3½ × 4⅛ × 3½	20.9	6.14	"	"	"	"	11/16	
	3/4	3⅜ × 4½ × 3⅜	22.9	6.75	"	"	"	"	3/4	
5	5/16	3½ × 5 × 3½	11.6	3.40	2½	1/2	1/2	2½	5/16	5
	3/8	3⅜ × 5⅛ × 3⅜	13.9	4.10	"	"	"	"	3/8	
	7/16	3⅜ × 5½ × 3⅜	15.4	4.51	"	"	"	"	7/16	
	1/2	3½ × 5 × 3½	17.3	5.25	"	"	"	"	1/2	
	9/16	3⅜ × 5⅛ × 3⅜	20.2	5.94	"	"	"	"	9/16	
	5/8	3⅜ × 5½ × 3⅜	22.6	6.64	"	"	"	"	5/8	
	11/16	3½ × 5 × 3½	23.7	6.96	"	"	"	"	11/16	
	3/4	3⅜ × 5⅛ × 3⅜	26.0	7.64	"	"	"	"	3/4	
	13/16	3½ × 5½ × 3½	28.0	8.23	"	"	"	"	13/16	
6	5/16	3½ × 6 × 3½	15.0	4.39	3½	1/2	1/2	3	5/16	6
	3/8	3⅜ × 6⅛ × 3⅜	18.3	5.39	"	"	"	"	3/8	
	7/16	3⅜ × 6½ × 3⅜	21.0	6.19	"	"	"	"	7/16	
	1/2	3½ × 6 × 3½	23.7	6.98	"	"	"	"	1/2	
	9/16	3⅜ × 6⅛ × 3⅜	26.4	7.48	"	"	"	"	9/16	
	5/8	3⅜ × 6½ × 3⅜	29.0	8.56	"	"	"	"	5/8	
	11/16	3½ × 6 × 3½	30.3	8.63	"	"	"	"	11/16	
	3/4	3⅜ × 6⅛ × 3⅜	33.0	9.40	"	"	"	"	3/4	
	7/8	3½ × 6½ × 3½	34.8	10.17	"	"	"	"	7/8	

WEIGHTS OF ANGLES

All dimensions in inches

SIZE	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	SIZE
8 x 8							26.4	29.5	32.7	35.8	38.9	42.0	45.0	48.0	51.0	54.0	56.9	8 x 8
6 x 6					14.8	17.2	19.6	21.9	24.2	26.5	28.7	30.9	33.1	35.3	37.4			6 x 6
* 5 x 5					12.3	14.3	16.2	18.1	20.0	21.8	23.6	25.4	27.2	28.9	30.6			* 5 x 5
4 x 4				8.2	9.8	11.3	12.8	14.3	15.7	17.1	18.5	19.9						4 x 4
3½ x 3½				7.1	8.5	9.8	11.1	12.3	13.6	14.8	16.0	17.1						3½ x 3½
3 x 3			4.9	6.1	7.2	8.3	9.4	10.4	11.4									3 x 3
* 2½ x 2½			4.5	5.5	6.6	7.6	8.5											* 2½ x 2½
2½ x 2½		3.1	4.0	5.0	5.9	6.8	7.7											2½ x 2½
* 2½ x 2½		2.8	3.7	4.5	5.3	6.1	6.8											* 2½ x 2½
2 x 2		2.5	3.2	4.0	4.7	5.3												2 x 2
1¾ x 1¾		2.1	2.8	3.4	4.0	4.6												1¾ x 1¾
1½ x 1½	1.2	1.8	2.4	2.9	3.4													1½ x 1½
1¼ x 1¼	1.0	1.5	1.9	2.4														1¼ x 1¼
1 x 1	0.8	1.2	1.5															1 x 1

SIZE	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	SIZE
* 7 x 3½						15.0	17.0	19.0	21.0	23.0	24.9	26.8	28.7	30.5	32.3			* 7 x 3½
6 x 4					12.3	14.3	16.2	18.1	20.0	21.8	23.6	25.4	27.2	28.9	30.6			6 x 4
6 x 3½					11.7	13.5	15.3	17.1	18.9	20.6	22.3	24.0	25.7	27.3	28.9			6 x 3½
* 5 x 4					11.0	12.8	14.5	16.2	17.8	19.5	21.1	22.6	24.2					* 5 x 4
5 x 3½				8.7	10.4	12.0	13.6	15.2	16.8	18.3	19.8	21.3	22.7					5 x 3½
5 x 3				8.2	9.8	11.3	12.8	14.2	15.7	17.1	18.5	19.9						5 x 3
* 4 x 3½				7.7	9.1	10.5	11.9	13.3	14.6	15.9	17.2	18.5						* 4 x 3½
4 x 3				7.1	8.5	9.8	11.1	12.3	13.6	14.8	16.0	17.1						4 x 3
3½ x 3				6.6	7.8	9.1	10.2	11.4	12.5	13.6	14.7	15.7						3½ x 3
3½ x 2½			4.9	6.1	7.2	8.3	9.4	10.4	11.4	12.4								3½ x 2½
3 x 2½			4.5	5.5	6.6	7.6	8.5	9.5										3 x 2½
3 x 2			4.0	5.0	5.9	6.8	7.7											3 x 2
2½ x 2		2.8	3.7	4.5	5.3	6.1	6.8											2½ x 2

SIZE	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	SIZE

Angles marked * are special

Carnegie Angles.
Areas in Square Inches.

ANGLES

Area in square inches.

SIZE	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	SIZE
8 x 8							7.75	8.68	9.61	10.53	11.44	12.34	13.23	14.12	15.00	15.87	16.73	8 x 8
6 x 6					4.36	5.06	5.75	6.43	7.11	7.78	8.44	9.09	9.74	10.37	11.00			6 x 6
*6 x 5					3.61	4.18	4.75	5.31	5.86	6.42	6.94	7.46	7.99	8.50	9.00			*5 x 5
4 x 4				2.40	2.86	3.31	3.75	4.18	4.61	5.03	5.44	5.84						4 x 4
3½ x 3½				2.09	2.48	2.87	3.25	3.62	3.98	4.34	4.69	5.03						3½ x 3½
3 x 3			1.44	1.78	2.11	2.43	2.75	3.06	3.36									3 x 3
*2½ x 2½			1.31	1.62	1.92	2.22	2.50											*2½ x 2½
2½ x 2½		0.90	1.19	1.47	1.73	2.00	2.25											2½ x 2½
*2½ x 2½		0.81	1.06	1.31	1.55	1.78	2.00											*2½ x 2½
2 x 2		0.72	0.94	1.15	1.36	1.56												2 x 2
1¾ x 1¾		0.62	0.81	1.00	1.17	1.30												1¾ x 1¾
1½ x 1½	0.36	0.53	0.69	0.84	0.99													1½ x 1½
1¼ x 1¼	0.30	0.43	0.56	0.69														1¼ x 1¼
1 x 1	0.24	0.34	0.44															1 x 1

SIZE	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	SIZE
*7 x 3½						4.40	5.00	5.59	6.17	6.75	7.31	7.87	8.42	8.97	9.50			*7 x 3½
6 x 4					3.61	4.18	4.75	5.31	5.86	6.41	6.94	7.47	7.99	8.50	9.00			6 x 4
6 x 3½					3.42	3.97	4.50	5.03	5.55	6.06	6.56	7.06	7.55	8.03	8.50			6 x 3½
*5 x 4					3.23	3.75	4.25	4.75	5.23	5.72	6.19	6.65	7.11					*5 x 4
5 x 3½				2.56	3.05	3.53	4.00	4.47	4.92	5.37	5.81	6.25	6.67					5 x 3½
5 x 3				2.40	2.86	3.31	3.75	4.18	4.61	5.03	5.44	5.84						5 x 3
*4 x 3½				2.25	2.67	3.09	3.50	3.90	4.30	4.68	5.06	5.43						*4 x 3½
4 x 3				2.09	2.48	2.87	3.25	3.62	3.98	4.34	4.69	5.03						4 x 3
3½ x 3				1.93	2.30	2.65	3.00	3.34	3.67	4.00	4.31	4.62						3½ x 3
3½ x 2½			1.44	1.78	2.11	2.43	2.75	3.06	3.36	3.65								3½ x 2½
3 x 2½			1.31	1.62	1.92	2.22	2.50	2.78										3 x 2½
3 x 2			1.19	1.47	1.73	2.00	2.25											3 x 2
2½ x 2		0.81	1.06	1.31	1.55	1.78	2.00											2½ x 2

SIZE	1/8	3/16	1/4	5/16	3/8	7/16	1/2	9/16	5/8	11/16	3/4	13/16	7/8	15/16	1	1 1/16	1 1/8	SIZE

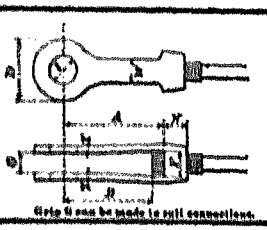
Angles marked * are special.

Upsets for Round and Square Bars.

ROUND ○ BARS							SQUARE □ BARS						
ROUND		UPSET					UPSET					SQUARE	
DIAM.	AREA	DIAM.	LENGTH	ADD	AREA AT ROOT	EXCESS AREA	EXCESS AREA	AREA AT ROOT	ADD	LENGTH	DIAM.	AREA	DIAM.
INCHES	SQ. INS.	INCHES	INCHES	INCHES	SQ. INS.	%	%	SQ. INS.	INCHES	INCHES	INCHES	SQ. INS.	INCHES
$\frac{8}{8}$	0.307	$\frac{7}{8}$	4	$4\frac{1}{2}$	0.420	36.8							$\frac{8}{8}$
$\frac{9}{8}$	0.442	1	4	$3\frac{7}{8}$	0.550	24.4	20.6	0.694	$3\frac{1}{2}$	4	$1\frac{1}{8}$	0.563	$\frac{9}{8}$
$\frac{7}{8}$	0.601	$1\frac{1}{8}$	4	5	0.691	48.3	16.3	0.891	4	4	$1\frac{1}{4}$	0.766	$\frac{7}{8}$
1	0.785	$1\frac{1}{8}$	4	$4\frac{1}{2}$	1.057	34.7	29.5	1.295	4	4	$1\frac{1}{3}$	1.000	1
$1\frac{1}{8}$	0.994	$1\frac{1}{4}$	4	$3\frac{7}{8}$	1.295	30.3	19.7	1.515	$4\frac{1}{2}$	$4\frac{1}{2}$	$1\frac{3}{8}$	1.266	$1\frac{1}{8}$
$1\frac{1}{4}$	1.227	$1\frac{1}{4}$	$4\frac{1}{2}$	$3\frac{7}{8}$	1.515	23.5	31.1	2.049	$4\frac{1}{2}$	$4\frac{1}{2}$	$1\frac{7}{8}$	1.563	$1\frac{1}{4}$
$1\frac{3}{8}$	1.485	$1\frac{3}{8}$	$4\frac{1}{2}$	$3\frac{1}{2}$	1.744	17.4	21.7	2.302	$4\frac{1}{2}$	5	2	1.891	$1\frac{3}{8}$
$1\frac{1}{2}$	1.767	2	5	$4\frac{1}{2}$	2.302	30.3	34.0	3.023	$4\frac{3}{4}$	5	$2\frac{1}{4}$	2.250	$1\frac{1}{2}$
$1\frac{5}{8}$	2.074	$2\frac{1}{8}$	5	$4\frac{1}{4}$	2.651	27.8	29.6	3.410	$4\frac{3}{4}$	$5\frac{1}{8}$	$2\frac{3}{8}$	2.641	$1\frac{5}{8}$
$1\frac{3}{4}$	2.405	$2\frac{1}{4}$	5	4	3.023	25.7	21.3	3.716	$4\frac{1}{4}$	$5\frac{1}{8}$	$2\frac{1}{8}$	3.063	$1\frac{3}{4}$
$1\frac{7}{8}$	2.761	$2\frac{3}{8}$	$5\frac{1}{4}$	$4\frac{1}{2}$	3.410	23.9	31.4	4.619	$5\frac{1}{8}$	6	$2\frac{5}{4}$	3.516	$1\frac{7}{8}$
2	3.142	$2\frac{1}{2}$	$5\frac{1}{2}$	$3\frac{7}{8}$	3.716	18.3	27.7	5.107	$4\frac{3}{4}$	6	$2\frac{7}{4}$	4.000	2
$2\frac{1}{8}$	3.547	$2\frac{1}{4}$	$5\frac{1}{2}$	$3\frac{3}{8}$	4.155	17.1	20.2	5.430	$4\frac{3}{8}$	6	3	4.516	$2\frac{1}{8}$
$2\frac{1}{4}$	3.976	$2\frac{3}{8}$	6	$4\frac{3}{8}$	5.107	28.5	28.6	6.510	$5\frac{1}{8}$	$6\frac{1}{8}$	$3\frac{1}{4}$	5.063	$2\frac{1}{4}$
$2\frac{3}{8}$	4.430	3	6	$4\frac{3}{8}$	5.430	22.6	33.8	7.548	$6\frac{1}{8}$	7	$3\frac{1}{8}$	5.641	$2\frac{3}{8}$
$2\frac{4}{8}$	4.909	$3\frac{1}{8}$	$6\frac{1}{4}$	$4\frac{3}{8}$	5.957	21.3	30.7	8.170	$6\frac{1}{4}$	8	$3\frac{1}{8}$	6.250	$2\frac{4}{8}$
$2\frac{5}{8}$	5.412	$3\frac{1}{4}$	$6\frac{1}{4}$	$4\frac{1}{4}$	6.510	20.3	35.0	9.305	$6\frac{3}{4}$	8	$3\frac{7}{8}$	6.891	$2\frac{5}{8}$
$2\frac{6}{8}$	5.940	$3\frac{3}{8}$	7	$4\frac{1}{4}$	7.088	19.3	32.1	9 994	6	8	4	7.563	$2\frac{6}{8}$
$2\frac{7}{8}$	6.492	$3\frac{1}{2}$	8	$5\frac{1}{2}$	8.170	25.9	37.0	11.329	8	9	$4\frac{1}{4}$	8.266	$2\frac{7}{8}$
3	7.069	$3\frac{1}{4}$	8	$5\frac{1}{4}$	8.641	22.2	41.7	12.753	$7\frac{1}{4}$	9	$4\frac{1}{2}$	9.000	3
$3\frac{1}{8}$	7.670	$3\frac{1}{8}$	8	$5\frac{1}{8}$	9,305	21.3							$3\frac{1}{8}$
$3\frac{1}{4}$	8.296	4	8	$4\frac{7}{8}$	9.994	20.7							$3\frac{1}{4}$
$3\frac{1}{8}$	9.621	$4\frac{1}{8}$	9	$5\frac{1}{2}$	11.329	17.7							$3\frac{1}{8}$
$3\frac{3}{4}$	11.045	$4\frac{1}{4}$	9	$4\frac{1}{4}$	12.753	15.5							$3\frac{3}{4}$

Clevises. American Bridge Company Standards.

All Dimensions in Inches.



Grip G can be made to suit connections.

Diam. of Clevis B	Max. Pin P	Clevis F	Clevis N	Clevis W	Clevis T	Clevis A	Clevis R	Diam. of Clevis D
3	1½	1½	1½	1½	½	6	5	3
4	2½	1½	1½	1½	½	9	8	4
5	3	2½	2½	2½	½	9	8	5
6	3½	2½	2½	3½	½	9	8	6
7	4	3½	3½	3½	½	9	8	7

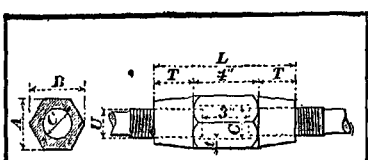
Table giving diameter of Clevis for given rod and pin.

Rod Round	Rod Square	Rod Upset	Pins 1	1¼	1½	1¾	2	2¼	2½	2¾	3	3¼	3½	3¾	4	Rod Upset	Rod Square	Rod Round
¾	⅝	1	3	3	3											1	⅞	¾
	1	1½	3	3	3	4	4	4								1½	1⅛	
⅞	1	1¼		4	4	4	4	4								1¼	1	⅞
1		1⅜		4	4	4	4	4								1⅜		1
1¼	1	1½		4	4	4	4	4	5	5	5					1½	1	1¼
1½	1½	1⅝			4	4	5	5	5	5	5					1⅝	1½	1½
1⅝		1⅝			5	5	5	5	5	5	5					1⅝		1⅝
	1⅝	1¾			5	5	5	5	5	5	5					1¾	1⅝	
1¾	1¾	2				5	5	5	5	5	6	6	6			2	1¾	1¾
1⅞		2⅛				5	5	5	5	5	6	6	6			2⅛		1⅞
1⅞	1⅞	2⅛					6	6	6	6	6	6	7	7	7	2⅛	1⅞	1⅞
1⅞	1⅞	2⅜					6	6	6	6	7	7	7	7	7	2⅜	1⅞	1⅞
2	1⅞	2⅜						6	6	7	7	7	7	7	7	2⅜	1⅞	2
2⅛		2⅝							7	7	7	7	7			2⅝		2⅛
	1⅞	2⅞							7	7	7	7	7			2⅞	1⅞	
2⅛	2	2⅛								7	7					2⅛	2	2⅛
Rod Round	Rod Square	Rod Upset	1	1¼	1½	1¾	2	2¼	2½	2¾	3	3¼	3½	3¾	4	Rod Upset	Rod Square	Rod Round
Rod			Pins													Rod		

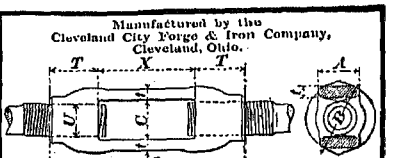
Clevises above and to right of heavy zigzag line, may be used with forks straight. Clevises below and to left of same line, should have forks closed in until pin is not overstrained.

Sleeve Nuts and Turnbuckles. American Bridge Company Standards.

All Dimensions in Inches.



Manufactured by the Cleveland City Forge & Iron Company, Cleveland, Ohio.



Standard Length, X = 0'
Extra Lengths, 9", 12", 18", 24", 30", 48", 72" (Special Prices).

Diam. of Screw U	Length of Thread T	Length of Nut L	Short Diam. A	Long Diam. B	Inside Diam. C	Thickness t	Weight in Lbs.	Weight in Lbs.	Standard Dimensions t	A	B	C	L	T	Diam. of Screw U
$\frac{7}{8}$	$1\frac{1}{2}$	7	$1\frac{5}{8}$	$1\frac{7}{8}$	$1\frac{1}{8}$	$\frac{1}{4}$	$2\frac{1}{2}$	$2\frac{1}{2}$	$\frac{5}{8}$	$1\frac{1}{4}$	$2\frac{1}{4}$	$1\frac{1}{2}$	$8\frac{5}{8}$	$1\frac{5}{16}$	$\frac{7}{8}$
1	$1\frac{1}{2}$	7	$1\frac{5}{8}$	$1\frac{7}{8}$	$1\frac{1}{8}$	$\frac{1}{4}$	3	$3\frac{1}{2}$	$\frac{7}{16}$	"	$2\frac{7}{16}$	$1\frac{5}{16}$	9	$1\frac{1}{8}$	1
$1\frac{1}{8}$	$1\frac{3}{4}$	$7\frac{1}{2}$	2	$2\frac{5}{16}$	$1\frac{5}{8}$	$\frac{5}{16}$	$3\frac{1}{2}$	4	$\frac{1}{2}$	"	$2\frac{9}{16}$	$1\frac{7}{16}$	$9\frac{5}{8}$	$1\frac{11}{16}$	$1\frac{1}{8}$
$1\frac{1}{4}$	"	"	"	"	"	"	4	$5\frac{1}{2}$	"	$1\frac{1}{2}$	$2\frac{3}{4}$	$1\frac{5}{8}$	$9\frac{5}{8}$	$1\frac{7}{8}$	$1\frac{1}{4}$
$1\frac{3}{8}$	2	8	$2\frac{3}{8}$	$2\frac{1}{4}$	$1\frac{5}{8}$	$\frac{3}{8}$	$4\frac{1}{2}$	6	"	$1\frac{5}{8}$	$3\frac{1}{16}$	$1\frac{11}{16}$	$10\frac{1}{8}$	$2\frac{1}{16}$	$1\frac{3}{8}$
$1\frac{1}{2}$	"	"	"	"	"	"	$6\frac{1}{2}$	7	$\frac{5}{8}$	"	$3\frac{3}{16}$	$1\frac{3}{4}$	$10\frac{1}{8}$	$2\frac{1}{4}$	$1\frac{1}{2}$
$1\frac{5}{8}$	$2\frac{1}{4}$	$8\frac{1}{2}$	$2\frac{3}{4}$	$3\frac{3}{16}$	$1\frac{3}{8}$	$\frac{7}{16}$	8	$8\frac{1}{2}$	"	$1\frac{3}{4}$	$3\frac{1}{4}$	2	$10\frac{3}{4}$	$2\frac{7}{16}$	$1\frac{5}{8}$
$1\frac{3}{4}$	"	"	"	"	"	"	$8\frac{1}{2}$	10	"	2	$3\frac{5}{8}$	$2\frac{1}{8}$	$11\frac{1}{2}$	$2\frac{5}{8}$	$1\frac{3}{4}$
$1\frac{7}{8}$	$2\frac{1}{2}$	9	$3\frac{1}{8}$	$3\frac{5}{8}$	$2\frac{1}{8}$	$\frac{1}{2}$	10	$11\frac{1}{2}$	$\frac{11}{16}$	"	$3\frac{7}{8}$	$2\frac{3}{16}$	$11\frac{5}{8}$	$2\frac{13}{16}$	$1\frac{7}{8}$
2	"	"	"	"	"	"	11	13	"	$2\frac{1}{4}$	$4\frac{1}{4}$	$2\frac{5}{8}$	12	3	2
$2\frac{1}{8}$	$2\frac{3}{4}$	$9\frac{1}{2}$	$3\frac{1}{8}$	$4\frac{1}{16}$	$2\frac{3}{8}$	$\frac{9}{16}$	14	15	$\frac{23}{32}$	$2\frac{1}{8}$	$4\frac{1}{8}$	$2\frac{1}{8}$	$12\frac{5}{8}$	$3\frac{3}{16}$	$2\frac{1}{8}$
$2\frac{1}{4}$	"	"	"	"	"	"	15	18	$\frac{13}{16}$	"	$4\frac{3}{8}$	$2\frac{11}{16}$	$12\frac{3}{4}$	$3\frac{3}{8}$	$2\frac{1}{4}$
$2\frac{3}{8}$	3	10	$3\frac{3}{8}$	$4\frac{1}{2}$	$2\frac{5}{8}$	$\frac{5}{8}$	18	20	"	$2\frac{3}{4}$	$4\frac{7}{8}$	$2\frac{3}{4}$	$13\frac{1}{2}$	$3\frac{9}{16}$	$2\frac{3}{8}$
$2\frac{1}{2}$	"	"	"	"	"	"	19	24	$\frac{27}{32}$	3	$5\frac{1}{8}$	$3\frac{1}{16}$	$13\frac{1}{2}$	$3\frac{3}{4}$	$2\frac{1}{2}$
$2\frac{5}{8}$	$3\frac{1}{4}$	$10\frac{1}{2}$	$4\frac{1}{4}$	$4\frac{15}{16}$	$2\frac{7}{8}$	$\frac{11}{16}$	22	28	$\frac{7}{8}$	"	$5\frac{1}{8}$	$3\frac{1}{8}$	$13\frac{7}{8}$	$3\frac{15}{16}$	$2\frac{5}{8}$
$2\frac{3}{4}$	"	"	"	"	"	"	23	30	"	$3\frac{1}{4}$	$5\frac{3}{4}$	$3\frac{1}{4}$	$14\frac{1}{4}$	$4\frac{1}{8}$	$2\frac{3}{4}$
$2\frac{7}{8}$	$3\frac{1}{2}$	11	$4\frac{1}{8}$	$5\frac{3}{8}$	$3\frac{1}{8}$	$\frac{3}{4}$	27	34	$1\frac{1}{32}$	"	$6\frac{1}{16}$	$3\frac{7}{16}$	$14\frac{5}{8}$	$4\frac{5}{16}$	$2\frac{7}{8}$
3	"	"	"	"	"	"	28	38	"	$3\frac{1}{2}$	$6\frac{1}{8}$	$3\frac{5}{8}$	15	$4\frac{1}{8}$	3
$3\frac{1}{8}$	$3\frac{3}{4}$	$11\frac{1}{2}$	5	$5\frac{13}{16}$	$3\frac{3}{8}$	$\frac{13}{16}$	34								$3\frac{1}{8}$
$3\frac{1}{4}$	"	"	"	"	"	"	35	50	$1\frac{1}{16}$	4	$6\frac{3}{4}$	$3\frac{7}{8}$	$15\frac{1}{2}$	$4\frac{7}{8}$	$3\frac{1}{4}$
$3\frac{3}{8}$	4	12	$5\frac{3}{8}$	$6\frac{1}{8}$	$3\frac{5}{8}$	$\frac{7}{8}$	39								$3\frac{3}{8}$
$3\frac{1}{2}$	"	"	"	"	"	"	40	65	$1\frac{7}{32}$	4	$7\frac{1}{4}$	$4\frac{1}{4}$	$16\frac{1}{4}$	$5\frac{1}{4}$	$3\frac{1}{2}$
$3\frac{5}{8}$	$4\frac{1}{4}$	$12\frac{1}{2}$	$5\frac{3}{4}$	$6\frac{11}{16}$	$3\frac{7}{8}$	$\frac{15}{16}$	45								$3\frac{5}{8}$
$3\frac{3}{4}$	"	"	"	"	"	"	47		$1\frac{1}{16}$	5	$8\frac{1}{4}$	$4\frac{7}{16}$	18	6	$3\frac{3}{4}$
$3\frac{7}{8}$	$4\frac{1}{2}$	13	$6\frac{1}{8}$	$7\frac{1}{8}$	$4\frac{1}{8}$	1	52								$3\frac{7}{8}$
4	"	"	"	"	"	"	55		$1\frac{7}{16}$	5	$8\frac{3}{4}$	$4\frac{5}{8}$	18	6	4
$4\frac{1}{4}$	$4\frac{1}{2}$	$13\frac{1}{2}$	$6\frac{1}{2}$	$7\frac{9}{16}$	$4\frac{3}{8}$	$1\frac{1}{16}$	65								$4\frac{1}{4}$
$4\frac{1}{2}$	5	14	$6\frac{7}{8}$	8	$4\frac{3}{4}$	$1\frac{1}{16}$	75								$4\frac{1}{2}$

Loop Bars. American Bridge Company Standards.

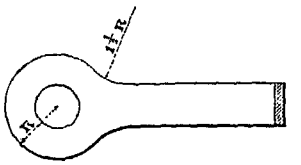
Length in inches beyond pin centre to form one eye equals 3.1 (P + 2)

Diam. of Bars	Diam. of Pins	DIAM. OF SIDE OF BARS																								Diam. of Pins
	2	2½	3	4	4½	5	5½	6	6½	7	7½	8	8½	9	9½	10	10½	11	11½	12	12½	13	13½	14	14½	
1	11½	12	12½	13	13½	14	14½	15	15½	16	16½	17	17½	18	18½	19	19½	20	20½	21	21½	22	22½	23	23½	1
1¼	12½	13	13½	14	14½	15	15½	16	16½	17	17½	18	18½	19	19½	20	20½	21	21½	22	22½	23	23½	24	24½	1¼
1½	13½	13½	14	14½	15	15½	16	16½	17	17½	18	18½	19	19½	20	20½	21	21½	22	22½	23	23½	24	24½	25	1½
1¾	14½	14½	15	15½	16	16½	17	17½	18	18½	19	19½	20	20½	21	21½	22	22½	23	23½	24	24½	25	25½	26	1¾
2	15½	16	16½	17	17½	18	18½	19	19½	20	20½	21	21½	22	22½	23	23½	24	24½	25	25½	26	26½	27	27½	2
2¼	16½	17	17½	18	18½	19	19½	20	20½	21	21½	22	22½	23	23½	24	24½	25	25½	26	26½	27	27½	28	28½	2¼
2½	17½	18	18½	19	19½	20	20½	21	21½	22	22½	23	23½	24	24½	25	25½	26	26½	27	27½	28	28½	29	29½	2½
2¾	18½	18½	19	19½	20	20½	21	21½	22	22½	23	23½	24	24½	25	25½	26	26½	27	27½	28	28½	29	29½	30	2¾
3	19½	19½	20	20½	21	21½	22	22½	23	23½	24	24½	25	25½	26	26½	27	27½	28	28½	29	29½	30	30½	31	3
3¼	20½	20½	21	21½	22	22½	23	23½	24	24½	25	25½	26	26½	27	27½	28	28½	29	29½	30	30½	31	31½	32	3¼
3½	21½	21½	22	22½	23	23½	24	24½	25	25½	26	26½	27	27½	28	28½	29	29½	30	30½	31	31½	32	32½	33	3½
3¾	22½	22½	23	23½	24	24½	25	25½	26	26½	27	27½	28	28½	29	29½	30	30½	31	31½	32	32½	33	33½	34	3¾
4	23½	23½	24	24½	25	25½	26	26½	27	27½	28	28½	29	29½	30	30½	31	31½	32	32½	33	33½	34	34½	35	4
4¼	24½	24½	25	25½	26	26½	27	27½	28	28½	29	29½	30	30½	31	31½	32	32½	33	33½	34	34½	35	35½	36	4¼
4½	25½	25½	26	26½	27	27½	28	28½	29	29½	30	30½	31	31½	32	32½	33	33½	34	34½	35	35½	36	36½	37	4½
4¾	26½	26½	27	27½	28	28½	29	29½	30	30½	31	31½	32	32½	33	33½	34	34½	35	35½	36	36½	37	37½	38	4¾
5	26½	26½	27	27½	28	28½	29	29½	30	30½	31	31½	32	32½	33	33½	34	34½	35	35½	36	36½	37	37½	38	5
5¼	27½	27½	28	28½	29	29½	30	30½	31	31½	32	32½	33	33½	34	34½	35	35½	36	36½	37	37½	38	38½	39	5¼
5½	28½	28½	29	29½	30	30½	31	31½	32	32½	33	33½	34	34½	35	35½	36	36½	37	37½	38	38½	39	39½	40	5½
5¾	29½	29½	30	30½	31	31½	32	32½	33	33½	34	34½	35	35½	36	36½	37	37½	38	38½	39	39½	40	40½	41	5¾
6	30	30½	31	31½	32	32½	33	33½	34	34½	35	35½	36	36½	37	37½	38	38½	39	39½	40	40½	41	41½	42	6

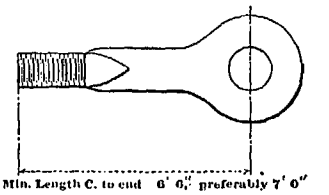
NOTE. Maximum shipping length should not exceed 35 feet.

Eye Bars. American Bridge Company Standards.

Ordinary



Adjustable



Min. Length C. to end 6′ 0″ preferably 7′ 0″

WIDTH OF BAR	MIN. THICKNESS OF BAR	HEAD			SCREW END			THICKNESS OF BAR	WIDTH OF BAR
		DIAM.	MAX. PIN.	ADD'L MATERIAL FOR HEAD	ADD'L MATERIAL FOR UPSET	DIAM.	LENGTH		
INS.	INS.	INS.	INS.	FT. & INS.	FT. & INS.	INS.	INS.	INS.	INS.
2	$\frac{1}{2}$	$4\frac{1}{2}$	$1\frac{1}{2}$	0 - $7\frac{1}{2}$	0 - 7	2	5	$\frac{15}{16}$	2
	"	$5\frac{1}{2}$	$2\frac{1}{4}$	1 - $0\frac{1}{2}$					
$2\frac{1}{2}$	$\frac{1}{4}$	$5\frac{1}{2}$	$2\frac{1}{2}$	0 - $9\frac{1}{2}$	1 - 1	$2\frac{1}{4}$	5	$\frac{15}{16}$ to $1\frac{1}{16}$	$2\frac{1}{2}$
	"	$6\frac{1}{2}$	$3\frac{1}{4}$	1 - $1\frac{1}{2}$					
3	$\frac{5}{8}$	7	3	1 - 3	1 - 5	$2\frac{1}{2}$	$5\frac{1}{2}$	1 to $1\frac{1}{16}$	3
	"	8	4	1 - 6	1 - 5	$2\frac{3}{4}$	6	$1\frac{1}{8}$ to $1\frac{1}{4}$	
4	$\frac{3}{4}$	$9\frac{1}{2}$	$4\frac{1}{4}$	1 - 8	1 - 8	3	6	1 to $1\frac{1}{8}$	4
	"	$10\frac{1}{2}$	$5\frac{1}{4}$	1 - 10	1 - 8	$3\frac{1}{4}$	$6\frac{1}{2}$	$1\frac{1}{16}$ to $1\frac{1}{4}$	
5	$\frac{7}{8}$	$11\frac{1}{2}$	5	1 - 9	1 - 9	$3\frac{1}{2}$	$6\frac{1}{2}$	1 to $1\frac{1}{16}$	5
	1	$12\frac{1}{2}$	6	2 - 1	1 - 9	$3\frac{1}{2}$	7	$1\frac{1}{8}$ to $1\frac{1}{4}$	
6	$\frac{7}{8}$	$13\frac{1}{2}$	$5\frac{1}{2}$	1 - 11	1 - 11	$3\frac{3}{4}$	8	$1\frac{1}{8}$ to $1\frac{1}{16}$	6
	1	$14\frac{1}{2}$	$6\frac{1}{2}$	2 - 2	1 - 11	4	8	$1\frac{1}{8}$ to $1\frac{1}{4}$	
7	$\frac{7}{8}$	16	$6\frac{3}{4}$	2 - 3	2 - 3	$4\frac{1}{4}$	9	$1\frac{1}{8}$ to $1\frac{1}{16}$	7
	$1\frac{1}{8}$	17	$7\frac{1}{4}$	2 - 8	2 - 3	$4\frac{1}{2}$	9	$1\frac{3}{8}$ to $1\frac{1}{2}$	
8	1	17	$6\frac{1}{2}$	2 - 3					8
	$1\frac{1}{8}$	18	$7\frac{1}{2}$	2 - 6					
	$1\frac{1}{4}$	$18\frac{1}{4}$	8	2 - 10					
9	$1\frac{1}{4}$	$19\frac{1}{4}$	$7\frac{3}{4}$	2 - 6					9
	"	$21\frac{1}{4}$	$9\frac{3}{4}$	3 - 1					
10	$1\frac{3}{8}$	22	9	2 - 11					10
	"	23	10	3 - 3					
12									12

Note: Eye bars are hydraulic forged, and are guaranteed to develop the full strength of the bar, under conditions given in the above table, when tested to destruction.

Maximum Allowable Bending Moments in Pins for Various Fibre Stresses.

PIN		MOMENTS IN INCH POUNDS FOR FIBRE STRESSES PER SQ. IN. OF				
DIAM. INCHES	AREA	15,000	18,000	20,000	22,000	25,000
1	0.785	1470	1770	1960	2160	2450
1¼	1.227	2880	3450	3830	4220	4790
1½	1.767	4970	5960	6630	7290	8280
1¾	2.405	7890	9470	10500	11570	13200
2	3.142	11800	14100	15700	17280	19600
2¼	3.976	16800	20100	22400	24600	28000
2½	4.909	23000	27600	30700	33700	38400
2¾	5.940	30600	36800	40800	44900	51000
3	7.069	39800	47700	53000	58300	66300
3¼	8.296	50600	60700	67400	74100	84300
3½	9.621	63100	75800	84200	92600	105200
3¾	11.045	77700	93200	103500	113900	129400
4	12.566	94200	113100	125700	138200	157100
4¼	14.186	113000	135700	150700	165800	188400
4½	15.904	134200	161000	178900	196800	223700
4¾	17.721	157800	189400	210400	231500	263000
5	19.635	184100	220900	245400	270000	306800
5¼	21.648	213100	255700	284100	312500	355200
5½	23.758	245000	294000	326700	359300	408300
5¾	25.967	280000	335900	373300	410600	466600
6	28.274	318100	381700	424100	466500	530200
6¼	30.680	359500	431400	479400	527300	599200
6½	33.183	404400	485300	539200	593100	674000
6¾	35.785	452900	543500	603900	664200	754800
7	38.485	505100	606100	673500	740800	841900
7¼	41.282	561200	673400	748200	823000	935300
7½	44.179	621300	745500	828400	911200	1035400
7¾	47.173	685500	822600	914000	1005300	1142500
8	50.265	754000	904800	1005300	1105800	1256600
8¼	53.456	826900	992300	1102500	1212800	1378200
8½	56.745	904400	1085200	1205800	1326400	1507300
8¾	60.132	986500	1183800	1315400	1446900	1644200

Table for Rivet Spacing.

Gauge	Diameter of Rivet															Gauge
	⅛	⅝	¼	⅞	⅜	⅞	½	⅝	⅝	¾	⅞	⅞	1	1⅛	1¼	
1																1
2	·3½	·3½	·3½	·3½	·3½	·3½	·3½	·4	·4½	·4½	·4½	·5	·5½	·5½	·5½	2
3	·3½	·3½	·4½	·4½	·4½	·5½	·5½	·6	·6½	·6½	·7½	·7½	·7½	·8½	·8½	3
4	·4½	·5	·5½	·6	·6½	·7	·7½	·8	·8½	·8½	·9	·9½	·10	·10½	·11	4
5	·5½	·6½	·6½	·7½	·8½	·8½	·9½	·10	·10½	·11½	1·0	1·0½	1·1	1·1½	1·2	5
6	·6½	·7½	·8½	·9	·9½	·10½	·11½	1·0	1·0½	1·1½	1·2	1·2½	1·3	1·3½	1·4	6
7	·7½	·8½	·9½	·10½	·11½	1·0½	1·1½	1·2	1·2½	1·3½	1·4	1·4½	1·5	1·5½	1·6	7
8	·9	·10	·11	1·0	1·1	1·2	1·3	1·4	1·4½	1·5½	1·6	1·6½	1·7	1·7½	1·8	8
9	·10½	·11½	1·0½	1·1½	1·2½	1·3½	1·4½	1·5	1·5½	1·6½	1·7½	1·8	1·8½	1·9	1·9½	9
10	·11½	1·0½	1·1½	1·2½	1·3½	1·4½	1·5½	1·6	1·6½	1·7½	1·8½	1·9	1·9½	2·0	2·0½	10
11	1·0½	1·1½	1·2½	1·3½	1·4½	1·5½	1·6½	1·7	1·7½	1·8½	1·9½	2·0	2·0½	2·1	2·1½	11
12	1·1½	1·2½	1·3½	1·4½	1·5½	1·6½	1·7½	1·8	1·8½	1·9½	2·0½	2·1	2·1½	2·2	2·2½	12
13	1·2½	1·3½	1·4½	1·5½	1·6½	1·7½	1·8½	1·9	1·9½	2·0½	2·1½	2·2	2·2½	2·3	2·3½	13
14	1·3½	1·4½	1·5½	1·6½	1·7½	1·8½	1·9½	2·0	2·0½	2·1½	2·2½	2·3	2·3½	2·4	2·4½	14
15	1·4½	1·5½	1·6½	1·7½	1·8½	1·9½	2·0½	2·1	2·1½	2·2½	2·3½	2·4	2·4½	2·5	2·5½	15
16	1·5½	1·6½	1·7½	1·8½	1·9½	2·0½	2·1½	2·2	2·2½	2·3½	2·4½	2·5	2·5½	2·6	2·6½	16
17	1·6½	1·7½	1·8½	1·9½	2·0½	2·1½	2·2½	2·3	2·3½	2·4½	2·5½	2·6	2·6½	2·7	2·7½	17
18	1·7½	1·8½	1·9½	2·0½	2·1½	2·2½	2·3½	2·4	2·4½	2·5½	2·6½	2·7	2·7½	2·8	2·8½	18
19	1·8½	1·9½	2·0½	2·1½	2·2½	2·3½	2·4½	2·5	2·5½	2·6½	2·7½	2·8	2·8½	2·9	2·9½	19
20	1·9½	2·0½	2·1½	2·2½	2·3½	2·4½	2·5½	2·6	2·6½	2·7½	2·8½	2·9	2·9½	3·0	3·0½	20
21	2·0½	2·1½	2·2½	2·3½	2·4½	2·5½	2·6½	2·7	2·7½	2·8½	2·9½	3·0	3·0½	3·1	3·1½	21
22	2·1½	2·2½	2·3½	2·4½	2·5½	2·6½	2·7½	2·8	2·8½	2·9½	3·0½	3·1	3·1½	3·2	3·2½	22
23	2·2½	2·3½	2·4½	2·5½	2·6½	2·7½	2·8½	2·9	2·9½	3·0½	3·1½	3·2	3·2½	3·3	3·3½	23
24	2·3½	2·4½	2·5½	2·6½	2·7½	2·8½	2·9½	3·0	3·0½	3·1½	3·2½	3·3	3·3½	3·4	3·4½	24
25	2·4½	2·5½	2·6½	2·7½	2·8½	2·9½	3·0½	3·1	3·1½	3·2½	3·3½	3·4	3·4½	3·5	3·5½	25
26	2·5½	2·6½	2·7½	2·8½	2·9½	3·0½	3·1½	3·2	3·2½	3·3½	3·4½	3·5	3·5½	3·6	3·6½	26
27	2·6½	2·7½	2·8½	2·9½	3·0½	3·1½	3·2½	3·3	3·3½	3·4½	3·5½	3·6	3·6½	3·7	3·7½	27
28	2·7½	2·8½	2·9½	3·0½	3·1½	3·2½	3·3½	3·4	3·4½	3·5½	3·6½	3·7	3·7½	3·8	3·8½	28
29	2·8½	2·9½	3·0½	3·1½	3·2½	3·3½	3·4½	3·5	3·5½	3·6½	3·7½	3·8	3·8½	3·9	3·9½	29
30	2·9½	3·0½	3·1½	3·2½	3·3½	3·4½	3·5½	3·6	3·6½	3·7½	3·8½	3·9	3·9½	4·0	4·0½	30
Gauge	⅛	⅝	¼	⅞	⅜	⅞	½	⅝	⅝	¾	⅞	⅞	1	1⅛	1¼	Gauge
	Diameter of Rivet															

Table for Rivet Spacing.

Spaces	Pitch in inches															Spaces
	3	3¼	3½	3¾	3¼	3⅞	4	4¼	4½	4¾	5	5¼	5½	5¾	6	
1																1
2	-6	-6¼	-6½	-6¾	-7	-7¼	-8	-8¼	-9	-9¼	-10	-10¼	-11	-11¼	1-0	2
3	-9	-9¾	-10¼	-10¾	-11	1-0	1-0	1-0¾	1-1¼	1-2¼	1-3	1-3¾	1-4¼	1-5¼	1-6	3
4	1-0	1-0¼	1-1	1-1¼	1-2	1-3	1-4	1-5	1-6	1-7	1-8	1-9	1-10	1-11	2-0	4
5	1-3	1-3¾	1-4¼	1-4¾	1-5¼	1-6¾	1-8	1-9¾	1-10¼	1-11¾	2-1	2-2¼	2-3¼	2-4¾	2-6	5
6	1-6	1-6¼	1-7¼	1-8¼	1-9	1-10¼	2-0	2-1¼	2-3	2-4¼	2-6	2-7¼	2-9	2-10¼	3-0	6
7	1-9	1-9¾	1-10¾	1-11¾	2-0¼	2-2¼	2-4	2-5¾	2-7¼	2-9¼	2-11	3-0¾	3-2¼	3-4¼	3-6	7
8	2-0	2-1	2-2	2-3	2-4	2-6	2-8	2-10	3-0	3-2	3-4	3-6	3-8	3-10	4-0	8
9	2-3	2-4¼	2-5¼	2-6¾	2-7½	2-9¾	3-0	3-2¼	3-4½	3-6¾	3-9	3-11½	4-1½	4-3¾	4-6	9
10	2-6	2-7½	2-8½	2-9¾	2-11	3-1½	3-4	3-6¼	3-9	3-11½	4-2	4-4½	4-7	4-9½	5-0	10
11	2-9	2-10¾	2-11¾	3-1¼	3-2¾	3-5¼	3-8	3-10¾	4-1¼	4-4¼	4-7	4-9¾	5-0¼	5-3¼	5-6	11
12	3-0	3-1¼	3-3	3-4¼	3-6	3-9	4-0	4-3	4-6	4-9	5-0	5-3	5-6	5-9	6-0	12
13	3-3	3-4¼	3-6¼	3-7¾	3-9¼	4-0¼	4-4	4-7¼	4-10¼	5-1¼	5-5	5-8¼	5-11½	6-2¼	6-6	13
14	3-6	3-7½	3-9¼	3-11½	4-1	4-4¼	4-8	4-11¼	5-3	5-6¼	5-10	6-1¼	6-5	6-8¼	7-0	14
15	3-9	3-10¾	4-0¼	4-2¼	4-4½	4-8¼	5-0	5-3¼	5-7½	5-11¼	6-3	6-6¾	6-10½	7-2½	7-6	15
16	4-0	4-2	4-4	4-6	4-8	5-0	5-4	5-8	6-0	6-4	6-8	7-0	7-4	7-8	8-0	16
17	4-3	4-5¼	4-7½	4-9¾	4-11½	5-3¾	5-8	6-0¼	6-4½	6-8¾	7-1	7-5¼	7-9¼	8-1½	8-6	17
18	4-6	4-8¼	4-10½	5-0¾	5-3	5-7½	6-0	6-4½	6-9	7-1½	7-6	7-10½	8-3	8-7½	9-0	18
19	4-9	4-11⅛	5-1½	5-4½	5-6½	5-11½	6-4	6-8½	7-1½	7-6½	7-11	8-3½	8-8½	9-1½	9-6	19
20	5-0	5-2½	5-5	5-7½	5-10	6-3	6-8	7-1	7-5	7-11	8-4	8-9	9-2	9-7	10-0	20
21	5-3	5-5¾	5-8¼	5-10¾	6-1¼	6-6¼	7-0	7-5¼	7-10½	8-3¾	8-9	9-2¼	9-7½	10-0¾	10-6	21
22	5-6	5-8¾	5-11½	6-2¼	6-5	6-10½	7-4	7-9¼	8-3	8-8¼	9-2	9-7½	10-1	10-6¼	11-0	22
23	5-9	5-11⅞	6-2½	6-5¾	6-8½	7-2½	7-8	8-1½	8-7½	9-1½	9-7	10-0¾	10-6½	11-0¾	11-6	23
24	6-0	6-3	6-6	6-9	7-0	7-6	8-0	8-6	9-0	9-6	10-0	10-6	11-0	11-6	12-0	24
25	6-3	6-6¼	6-9¼	7-0¾	7-3¼	7-9¼	8-4	8-10½	9-4½	9-10½	10-5	10-11½	11-5½	11-11½	12-6	25
26	6-6	6-9¼	7-0¼	7-3¾	7-7	8-1½	8-8	9-2½	9-9	10-3½	10-10	11-4½	11-11	12-5½	13-0	26
27	6-9	7-0¾	7-3½	7-7½	7-10½	8-5½	9-0	9-6¾	10-1½	10-8½	11-3	11-9¾	12-4½	12-11½	13-6	27
28	7-0	7-3½	7-7	7-10½	8-2	8-9	9-4	9-11	10-6	11-1	11-8	12-3	12-10	13-5	14-0	28
29	7-3	7-6¾	7-10½	8-1½	8-5½	9-0½	9-8	10-3½	10-10½	11-5½	12-1	12-8½	13-3½	13-10½	14-6	29
30	7-6	7-9½	8-1½	8-5½	8-9	9-4½	10-0	10-7½	11-3	11-10½	12-6	13-1½	13-9	14-4½	15-0	30
Spaces	3	3¼	3½	3¾	3¼	3⅞	4	4¼	4½	4¾	5	5¼	5½	5¾	6	Spaces
	Pitch in inches															

Table of Areas in Square Inches, to be Deducted from Riveted Plates or Shapes to Obtain Net Areas.

Thickness Plate in Inches.	Size of Hole. Inches.													
	¼	5/16	⅜	7/16	½	9/16	⅝	11/16	¾	13/16	⅞	15/16	1	1 1/16
1/4	.06	.08	.09	.11	.13	.14	.16	.17	.19	.20	.22	.23	.25	.27
5/16	.08	.10	.12	.14	.16	.18	.20	.21	.23	.25	.27	.29	.31	.33
3/8	.09	.12	.14	.16	.19	.21	.23	.26	.28	.30	.33	.35	.38	.40
7/16	.11	.14	.16	.19	.22	.25	.27	.30	.33	.36	.38	.41	.44	.46
1/2	.13	.16	.19	.22	.25	.28	.31	.34	.38	.41	.44	.47	.50	.53
9/16	.14	.18	.21	.25	.28	.32	.35	.39	.42	.46	.49	.53	.56	.60
5/8	.16	.20	.23	.27	.31	.35	.39	.43	.47	.51	.55	.59	.63	.66
11/16	.17	.21	.26	.30	.34	.39	.43	.47	.52	.56	.60	.64	.69	.73
3/4	.19	.23	.28	.33	.38	.42	.47	.52	.56	.61	.66	.70	.75	.80
13/16	.20	.25	.30	.36	.41	.46	.51	.56	.61	.66	.71	.76	.81	.86
7/8	.22	.27	.33	.38	.44	.49	.55	.60	.66	.71	.77	.82	.88	.93
15/16	.23	.29	.35	.41	.47	.53	.59	.64	.70	.76	.82	.88	.94	1.00
1	.25	.31	.38	.44	.50	.56	.63	.69	.75	.81	.88	.94	1.00	1.06
1 1/16	.27	.33	.40	.46	.53	.60	.66	.73	.80	.86	.91	1.00	1.06	1.13
1 1/8	.28	.35	.42	.49	.56	.63	.70	.77	.84	.91	.98	1.05	1.13	1.20
1 3/16	.30	.37	.45	.52	.59	.67	.74	.82	.89	.96	1.04	1.11	1.19	1.26
1 1/4	.31	.39	.47	.55	.63	.70	.78	.86	.94	1.02	1.09	1.17	1.25	1.33
1 5/16	.33	.41	.49	.57	.66	.74	.82	.90	.98	1.07	1.15	1.23	1.31	1.39
1 3/8	.34	.43	.52	.60	.69	.77	.86	.95	1.03	1.12	1.20	1.29	1.38	1.46
1 7/16	.36	.45	.54	.63	.72	.81	.90	.99	1.08	1.17	1.26	1.35	1.44	1.53
1 1/2	.38	.47	.56	.66	.75	.84	.94	1.03	1.13	1.22	1.31	1.41	1.50	1.59
1 9/16	.39	.49	.59	.68	.78	.88	.98	1.07	1.17	1.27	1.37	1.46	1.56	1.66
1 5/8	.41	.51	.61	.71	.81	.91	1.02	1.12	1.22	1.33	1.42	1.52	1.63	1.73
1 11/16	.42	.53	.63	.74	.84	.95	1.05	1.16	1.27	1.37	1.47	1.58	1.69	1.79
1 3/4	.44	.55	.66	.77	.88	.98	1.09	1.20	1.31	1.42	1.53	1.64	1.75	1.86
1 13/16	.45	.57	.68	.79	.91	1.02	1.13	1.25	1.36	1.47	1.99	1.70	1.81	1.93
1 7/8	.47	.59	.70	.82	.94	1.05	1.17	1.29	1.41	1.52	1.64	1.76	1.88	1.99
1 15/16	.48	.61	.73	.85	.97	1.09	1.21	1.33	1.45	1.57	1.70	1.82	1.94	2.06
2	.50	.63	.75	.88	1.00	1.13	1.25	1.38	1.50	1.63	1.75	1.88	2.00	2.13

In calculating the net area add ⅛ inch to diameter of rivet before entering the table.

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